CERTIFICATE

By Authority Of
THE UNITED STATES OF AMERICA
Legally Binding Document

By the Authority Vested By Part 5 of the United States Code § 552(a) and Part 1 of the Code of Regulations § 51 the attached document has been duly INCORPORATED BY REFERENCE and shall be considered legally binding upon all citizens and residents of the United States of America. *HEED THIS NOTICE*: Criminal penalties may apply for noncompliance.

* * *

Document Name: SEAC: Recommended Lateral Force Requirements and Commentary including Errata

CFR Section(s): 42 CFR 52b.12(c)(5)

Standards Body: Structural Engineers Association of California

Official Incorporator:
THE EXECUTIVE DIRECTOR
OFFICE OF THE FEDERAL REGISTER
WASHINGTON, D.C.

7ed 1P03 ~~98~~^{00}

35^{00}
Civil-
Seismology

RECOMMENDED LATERAL FORC $35.00
[illegible] STRUCTURAL ENGINEERS USED 115
POWELL'S 04 CIVIL-SEISMOLOGY 4

Recommende
Lateral Forc
Requirements
and Commentary

1996
Sixth Edition

Seismology Committee
Structural Engineers Association of California

Recommended Lateral Force Requirements and Commentary

1996
Sixth Edition

Seismology Committee
Structural Engineers Association of California

Copyright

© 1996 Structural Engineers Association of California

All rights reserved. This publication or any part thereof must not be reproduced in any form without the written permission of the Structural Engineers Association of California.

Publisher

Structural Engineers Association of California (SEAOC)
555 University Avenue, Suite 126
Sacramento, California 95825-6510
Telephone: 916-427-3647
Fax: 916-568-0677

The Structural Engineers Association of California (SEAOC) is a professional association of four member organizations representing the structural engineering community in California. This document is published in keeping with SEAOC's stated mission "to advance the structural engineering profession; to provide the public with structures of dependable performance through the application of state of the art structural engineering principles; to assist the public in obtaining professional structural engineering services; to promote natural hazard mitigation; to provide continuing education and encourage research; to provide structural engineers with the most current information and tools to improve their practice; and to maintain the honor and dignity of the profession."

Editor

Gail Hynes Shea, Albany, California

Print History

First Edition, 1959
First Edition with Commentary, 1960
First Edition Revised, 1963
Second Edition without Commentary, 1966
Second Edition with Commentary, 1967
Second Edition with Commentary and Addendum, 1968
Third Edition with Commentary, 1973
Fourth Edition with Partial Commentary, 1974
Commentary for Fourth Edition, 1975
Fourth Edition Revised, 1980
Fifth Edition with Tentative Commentary, 1988
Fifth Edition Revised with Commentary, 1990
Sixth Edition with Commentary, 1996

Disclaimer

Neither SEAOC, nor its member organization, committees, writers, editors, or individuals who have contributed to this publication make any warranty, expressed or implied, or assume any legal liability or responsibility for the use, application of, and/or reference to opinions, findings, conclusions, or recommendations included in the publication

Table of Contents

List of Tables xv

List of Figures xvii

Preface xix

Changes in this Edition of the Blue Book xix

Summary of Significant Technical Changes xxi

Acknowledgments xxiii

Suggestions for Improvement xxix

Reference to 1994 UBC xxix

Requirements

Chapter 1

General Requirements for Design and Construction of Earthquake-Resistive Structures 1

101 UBC §1624 General 1
- 101.1 Scope and Objectives 1
- 101.2 Wind and Seismic Forces 1
- 101.3 Continuous Load Path 1
- 101.4 Basis for Seismic Design 1
- 101.5 Computer Calculations 2

102 UBC §1625 Definitions 2

103 UBC §1626 Symbols and Notations 5

104 UBC §1627 Criteria Selection 6
- 104.1 Basis for Design 6
- 104.2 Seismic Zones 6
- 104.3 Site Geology and Soil Characteristics 6
- 104.4 Occupancy Categories 6
- 104.5 Configuration Requirements 6
- 104.6 Structural Systems 7
- 104.7 Height Limits 8

104.8 Selection of Lateral Force Procedure 8
104.9 System Limitations 9
104.10 Alternative Procedures 9

105 UBC §1628 Minimum Design Lateral Forces and Related Effects 10
105.1 General 10
105.2 Static Force Procedure 10
105.3 Combinations of Structural Systems 11
105.4 Vertical Distribution of Force 12
105.5 Horizontal Distribution of Shear 13
105.6 Horizontal Torsional Moments 13
105.7 Overturning 14
105.8 Story Drift Limitation 15
105.9 P-delta Effects 15
105.10 Vertical Component of Seismic Forces 15

106 UBC §1629 Dynamic Lateral Force Procedure 16
106.1 General 16
106.2 Ground Motion 16
106.3 Mathematical Model 17
106.4 Dynamic Analysis Methods 17
106.5 Time-History Analysis 18

107 UBC §1630 Lateral Force on Elements of Structures, Nonstructural Components, and Equipment Supported by Structures 18
107.1 UBC §1630.1 General 18
107.2 UBC §1630.2 Total Design Seismic Force 19
107.3 UBC §1630.3 Specifying Lateral Forces 20
107.4 UBC §1630.4 Relative Motion of Equipment Attachments 20
107.5 UBC §1630.5 Alternative Designs 20

108 UBC §1631 Detailed Systems Design Requirements 21
108.1 General 21
108.2 Structural Framing Systems 22

109 UBC §1632 Nonbuilding Structures 26
109.1 General 26
109.2 Lateral Force 27
109.3 Rigid Structures 28
109.4 Tanks With Supported Bottoms 28
109.5 Other Nonbuilding Structures 28

110 UBC §3400 Existing Structures 29
110.1 UBC 3401 General 29
110.2 UBC §3402 Maintenance 29
110.3 UBC §3403 Additions, Alterations, or Repairs 29
110.4 UBC §3040 Moved Buildings 32
110.5 UBC §3405 Change In Use 32

111 Future Objectives and Research Requirements 32

112–149 Reserved 32

150 UBC §1650 General Requirements for Seismic-Isolated Structures 32

151 UBC §1651 Definitions for Seismic-Isolated Structures 32

152 UBC §1652 Symbols and Notations for Seismic-Isolated Structures 33

153 UBC §1653 Criteria Selection for Seismic-Isolated Structures 35

153.1 Basis for Design 35
153.2 Stability of the Isolation System 36
153.3 Occupancy Categories 36
153.4 Configuration Requirements 36
153.5 Selection of Lateral Response Procedure 36

154 UBC §1654 Static Lateral Response Procedure for Seismic-Isolated Structures 37
154.1 General 37
154.2 Deformation Characteristics of the Isolation System 37
154.3 Minimum Lateral Displacements 38
154.4 Minimum Lateral Forces 38
154.5 Vertical Distribution of Force 39
154.6 Drift Limits 39

155 UBC §1655 Dynamic Lateral Response Procedure for Seismic-Isolated Structures 40
155.1 General 40
155.2 Isolation System and Structural Elements at or Below the Isolation Interface 40
155.3 Structural Elements Above the Isolation System 40
155.4 Ground Motion 41
155.5 Mathematical Model for Seismic-Isolated Structures 42
155.6 Description of Analysis Procedures 43
155.7 Design Lateral Force 43
155.8 Drift Limits 44

156 UBC §1656 Lateral Load on Elements of Structures and Nonstructural Components Supported by Seismic-Isolated Structures 44
156.1 General 44
156.2 Forces and Displacements 44

157 UBC §1657 Detailed Systems Requirements for Seismic-Isolated Structures 45
157.1 General 45
157.2 Isolation System 45
157.3 Structural System 47

158 UBC §1658 Nonbuilding Seismic-Isolated Structures 47

159 UBC §1659 Foundations of Seismic-Isolated Structures 47

160 UBC §1660 Design and Construction Review for Seismic-Isolated Structures 47
160.1 General 47
160.2 Isolation System 47

161 UBC §1661 Required Tests of Isolation System for Seismic-Isolated Structures 48
161.1 General 48
161.2 Prototype Tests 48
161.3 Determination of Force-Deflection Characteristics 50
161.4 System Adequacy 50
161.5 Design Properties of the Isolation System 51

Appendix 1A Seismic Zone Coefficient 206

Appendix 1B C and S Coefficients 212

Appendix 1C **Development of the R_w Factor** 215
Appendix 1D **Dynamic Analysis** 221
Appendix 1E **Deformation Compatibility** 231

Chapter 2
Structural Tests and Inspections 65

201 **General Requirements** 65
202 **Recommended Modifications to 1994 UBC** 65
- 202.1 Special Inspections 65
- 202.2 Structural Observation 66
- 202.3 Design Review 66
- 202.4 Project Design Peer Review 66
- 202.5 Quality Assurance 66

Chapter 3
Foundations 67

301 **General Requirements** 67
302 **Recommended Modifications to 1994 UBC** 67
- 302.1 Allowable Foundation and Lateral Pressures 67
- 302.2 Requirements for Piles, Pile Caps, and Grade Beams in Seismic Zones 3 and 4 67

Chapter 4
Reinforced Concrete 71

401 **General Requirements** 71
402 **Recommended Modifications to 1994 UBC** 71
- 402.1 Details of Reinforcement 71
- 402.2 Strength Reduction Factors 71
- 402.3 Limits of Section 72
- 402.4 Circular Columns 72
- 402.5 Reinforced Concrete Structures Resisting Forces Induced by Earthquake Motions 72
- 402.6 Flexural Members of Frames 72
- 402.7 Frames Members Subjected to Bending and Axial Loads 72
- 402.8 Shear Strength 73
- 402.9 Design of Shear Walls for Flexural and Axial Loads 73
- 402.10 Boundaries of Structural Diaphragms 74
- 402.11 Coupling Beams 75
- 402.12 Minimum Thickness of Diaphragms 76
- 402.13 Frame Members Not Part of the Lateral Force Resisting System. . 76

Chapter 5
Lightweight Metals 79

501 **General Requirements** 79

Chapter 6
Reinforced Masonry. 81
601 General Requirements 81
602 Recommended Modifications to 1994 UBC. 81
  602.1 Standards of Quality. 81
  602.2 General Design Requirements. 81
  602.3 Design of Reinforced Masonry. 81
  602.4 Strength Design of Masonry. 82

Chapter 7
Structural Steel. 83
701 General Requirements 83
702 Recommended Modifications to 1994 UBC. 83

Chapter 8
Wood 85
801 General Requirements 85
802 Recommended Modifications to 1994 UBC. 85
  802.1 Safe Lateral Strength 85
  802.2 Wood Shear Walls and Diaphragms in Seismic Zones 3 and 4. . . 86
  802.3 Conventional Light Frame Construction. 86

Chapter 9
Other Materials. 89
901 General Requirements 89
902 Recommended Modifications to 1994 UBC. 89
903 Other Sheathing. 89

Commentary

Chapter 1
General Requirements for Design and Construction of Earthquake-Resistive Structures 93

C101 General 93
  C101.1 Scope and Objectives 93
  C101.2 Wind and Seismic Forces. 95
  C101.3 Continuous Load Path 96
  C101.4 Basis for Seismic Design 96
  C101.5 Computer Calculations. 96
C102 UBC §1625 Definitions 97
C103 UBC §1626 Symbols and Notations. 99
C104 UBC §1627 Criteria Selection. 99
  C104.1 Basis for Design 99
  C104.2 Seismic Zones 99
  C104.3 Site Geology and Soil Characteristics 100

C104.4 Occupancy Categories 101
C104.5 Configuration Requirements 101
C104.6 Structural Systems 104
C104.7 Height Limits 109
C104.8 Selection of Lateral Force Procedure 109
C104.9 System Limitations 113
C104.10 Alternative Procedures 115
C104.11 Types of Irregularity 115

C105 UBC §1628 Minimum Design Lateral Forces and Related Effects 118
C105.1 General 118
C105.2 Static Force Procedure 119
C105.3 Combinations of Structural Systems 127
C105.4 Vertical Distribution of Force 127
C105.5 Horizontal Distribution of Shear 127
C105.6 Horizontal Torsional Moments 128
C105.7 Overturning 129
C105.8 Story Drift Limitation 129
C105.9 P-delta Effects 130
C105.10 Vertical Component of Seismic Forces 131

C106 UBC §1629 Dynamic Lateral Force Procedure 131
C106.1 General 132
C106.2 Ground Motion 133
C106.3 Mathematical Model 136
C106.4 Dynamic Analysis Methods 139

C107 UBC §1630 Lateral Force on Elements of Structures, Nonstructural Components, and Equipment Supported by Structures 143
C107.1 General 145
C107.2 Total Design Seismic Force 152
C107.3 Specifying Lateral Forces 155
C107.4 Relative Motion of Equipment Attachments 155
C107.5 UBC §1630.5 Alternative Designs 156
C107.6 Additional Comments 157

C108 UBC §1631 Detailed Systems Design Requirements 157
C108.1 General 157
C108.2 Structural Framing Systems 159

C109 UBC §1632 Nonbuilding Structures 168
C109.1 General 168
C109.2 Lateral Force 170
C109.3 Rigid Structures 171
C109.4 No Commentary provided. 171
C109.5 Other Nonbuilding Structures 171

C110 UBC §3400 Existing Structures 176

C111 Future Objectives and Research Requirements 177

C112–C149 Reserved 177

C150 UBC §1650 General Requirements for Seismic-Isolated Structures . . . 178
C150.1 Introduction . . . 178
C150.2 General . . . 179
C151.1 Isolation System/Interface . . . 180
C151.2 Effective Stiffness and Damping . . . 180

C153 UBC §1653 Criteria Selection for Seismic-Isolated Structures . . . 181
C153.1 Basis for Design . . . 181
C153.2 Stability of the Isolation System . . . 181
C153.3 Occupancy Categories . . . 181
C153.4 Configuration Requirements . . . 181
C153.5 Selection of Lateral Response Procedure . . . 181

C154 UBC §1654 Static Lateral Response Procedure for Seismic-Isolated Structures . . . 182
C154.1 No Commentary provided . . . 182
C154.2 Deformational Characteristics of the Isolation System . . . 182
C154.3 Minimum Lateral Displacements . . . 183
C154.4 Minimum Lateral Forces . . . 184
C154.5 Vertical Distribution of Force . . . 185
C154.6 Drift Limits . . . 186

C155 UBC §1655 Dynamic Lateral Response Procedure for Seismic-Isolated Structures . . . 186
C155.1 General . . . 186
C155.2 Isolation System and Structural Elements at or Below the Isolation Interface . . . 186
C155.3 No Commentary provided . . . 186
C155.4 Ground Motion . . . 186
C155.5 Mathematical Model for Seismic-Isolated Structures . . . 187
C155.6 Description of Analysis Procedures . . . 188
C155.7 Design Lateral Force . . . 189

C156 No Commentary provided . . . 189

C157 UBC §1657 Detailed Systems Requirements for Seismic-Isolated Structures . . . 189
C157.1–C157.2.4 No Commentary provided . . . 189
C157.2.7–C157.2.8 No Commentary provided . . . 189

C158–C159 No Commentary provided . . . **190**

C160 UBC §1660 Design and Construction Review for Seismic-Isolated Structures . . . 190

C161 UBC §1661 Required Tests of Isolation System for Seismic-Isolated Structures . . . 190
C161.1 General . . . 190
C161.2 Prototype Tests . . . 190
C161.3 Determination of Force-Deflection Characteristics . . . 192
C161.4 No Commentary provided . . . 192
C161.5 Design Properties of the Isolation System . . . 192

Chapter 2
Structural Tests and Inspections. 243
C201 General Requirements. 243
C202 Recommended Modifications to the 1994 UBC. 243
C202.1 Special Inspections. 243
C202.2 Structural Observation 243
C202.3 Design Review 244
C202.4 Project Design Peer Review 244
C202.5 Quality Assurance. 245

Chapter 3
Foundations. 249
C301 General Requirements. 249
C302 Recommended Modifications to 1994 UBC 249
C302.1 Allowable Foundation and Lateral Pressures 249
C302.2 Requirements for Piles, Pile Caps, and Grade Beams in Seismic Zones 3 and 4 250

Chapter 4
Reinforced Concrete. 255
C401 General Requirements. 255
C402 Recommended Modifications to 1994 UBC 255
C402.1 Details of Reinforcement. 255
C402.2 Strength Reduction Factors 256
C402.3 Limits of Section 256
C402.4 Circular Columns 256
C402.5 Reinforced Concrete Structures Resisting Forces Induced by Earthquake Motions 256
C402.6 Flexural Members of Frames 257
C402.7 Frame Members Subjected to Bending and Axial Loads 257
C402.8 Shear Strength 257
C402.9 Design of Shear Walls for Flexural and Axial Loads. 257
C402.10 Boundaries of Structural Diaphragms. 257
C402.11 Coupling Beams 257
C402.12 Minimum Thickness of Diaphragms 258
C402.13 Frame Members Not Part of the Lateral Force Resisting System 258
C403 General Requirements. 258
C403.1 UBC §1921.2.3 Strength Reduction Factors 258
C403.2 UBC §1921.2.4.2 Lightweight Concrete. 259
C403.3 UBC §1921.2.5 Reinforcement in Members Resisting Earthquake Forces 259
C403.4 UBC §1921.2.7 Load Factors. 260
C404 Flexural Members of Frames. 261
C404.1 UBC §1921.3.1 Scope 261
C404.2 UBC §1921.3.2 Longitudinal Reinforcement 261

C404.3 UBC §1921.3.3 Transverse Reinforcement 262
C404.4 UBC §1921.3.3 Seismic Hooks 262
C404.5 UBC §1921.3.4.1 Shear Strength Design Forces 262
C405 Frame Members Subjected to Bending and Axial Loads. 262
C405.1 Scope 262
C405.2 UBC §1921.4.2 Minimum Flexural Strength of Columns 263
C405.3 UBC §1921.4.3.2 Splice Locations 263
C405.4 Transverse Reinforcement 263
C405.5 UBC §1921.4.5.1 Shear Strength Design Forces 264
C406 Joints of Frames 264
C406.1 UBC §1921.5.1 General Requirements 264
C406.2 UBC §1921.5.3.1 Shear Strength in Joint Region—Definition of Aj 266
C406.3 UBC §1921.5.4.1 Development Length for Reinforcement in Tension 266
C407 Shear Walls, Diaphragms, and Trusses. 266
C407.1 Reinforcement 267
C407.2 Design of Shear Walls for Flexural and Axial Loads 268
C407.3 UBC §1921.6.6.3 Boundaries of Structural Diaphragms 277
C407.4 UBC §1921.6.8 Discontinuous Walls 277
C407.5 Coupling Beams 277
C407.6 UBC §1921.6.10 Toppings on Precast Diaphragms 279
C407.7 UBC §1921.6.11 Minimum Thickness of Diaphragms 279
C407.8 UBC §1921.6.12 Wall Piers 280
C408 UBC §1921.7 Frame Members Not Part of the Lateral Force Resisting System 280
C408.1 Factored Gravity Forces 281
C408.2 Members Exceeding Design Strength 282
C409 UBC §1921.8 Requirements for Frames in Seismic Zone 2. 282
C410 Constructibility 282
C411 Design and Detailing Prerequisites 282

Chapter 5
Lightweight Metals. 289
C501 UBC §2001 General Requirements 289

Chapter 6
Reinforced Masonry. 291
C601 General Requirements 291
C602 Recommended Modifications to 1994 UBC. 291
C602.1 Standards of Quality. 291
C602.2 General Design Regulations 291
C602.3 Design of Reinforced Masonry. 291
C602.4 Strength Design of Masonry. 292
C603 UBC §2105 Quality Assurance. 292

C604 General Design Requirements 292
C604.1 UBC §2106.1.12.4 Special Provisions for Seismic Zones 3 and 4 292
C604.2 UBC §2106.2.14 Placement of Embedded Anchor Bolts 293
C604.3 Working Stress Design and Strength Design Requirements for Reinforced Masonry 293
C605 Working Stress Design of Masonry 293
C606 Strength Design of Masonry 294
C606.1 UBC §2108.1 General 294
C606.2 UBC §2108.1.2 Quality Assurance Provisions 294
C606.3 UBC §2108.1.4 Design Strength 294

Chapter 7
Structural Steel 301

C701 UBC §2211.1 General Requirements 301
C701.1 UBC §2211.2 Definitions 302
C702 Recommended Modifications to 1994 UBC 302
702.1 Prequalified Moment Connections 302
702.2 Connection Strength 302
C703 Steel Properties 303
C703.1 UBC §2211.4.1 Materials 303
C703.2 UBC §2211.4.2 Member Strength 304
C704 Column Requirements 305
C704.1 UBC §2211.5.1 Column Strength 305
C704.2 UBC §2211.5.2 Column Splices 306
C704.3 UBC §2211.5.3 Column Slenderness Evaluation 306
C705 UBC §2211.6 Ordinary Moment Frame Requirements 307
C706 UBC §2211.7 Special Moment Resisting Frame (SMRF) Requirements 307
C706.1 Girder-to-Column Connection 309
C706.2 Column-Girder Joint Panel Zone 310
C706.3 UBC §2211.7.3 Flange Width-Thickness Ratio 312
C706.4 UBC §2211.7.4 Continuity Plates 313
C706.5 UBC §2211.7.5 Strength Ratio 314
C706.6 UBC §2211.7.6 Trusses In SMRF 315
C706.7 UBC §2211.7.7 Girder-Column Joint Restraint 316
C706.8 UBC §2211.7.8 Beam Bracing 317
C706.9 UBC §2211.7.9 Changes in Beam Flange Area 317
C706.10 UBC §2211.7.10 Moment Frame Drift Calculations 318
C707 Requirements for Braced Frames 318
C707.1 UBC §2211.8.1 General 318
C707.2 Bracing Members 320
C707.3 Bracing Connection 321
C707.4 Bracing Configuration 322
C707.5 UBC §2211.8.5 One- and Two-Story Buildings 323
C708 Special Concentrically Braced Frames (SCBF) 323
C708.1 UBC §2211.9.1 General 323
C708.2 UBC §2211.9.2 Bracing Members 325

C708.3 UBC §2211.9.3 Bracing Connections 326
C708.4 UBC §2211.9.4 Bracing Configuration..................... 326
C708.5 UBC §2211.9.5 Columns.............................. 327
C708.6 UBC §2211.9.6 Nonbuilding Structures................... 327

C709 Eccentrically Braced Frame Requirements........................ 327

C709.1 UBC §2211.10.1 General 327
C709.2 UBC §2211.10.2 Link Beam 330
C709.3 UBC §2211.10.3 Link Beam Strength 330
C709.4 UBC §2211.10.4 Link Beam Rotation 331
C709.5 UBC §2211.10.5 Link Beam Web 332
C709.6 UBC §2211.10.6 Brace-Beam Connection 332
C709.7 UBC §2211.10.7 Link Beam Stiffeners 332
C709.8 UBC §2211.10.8 Intermediate Stiffeners.................. 332
C709.9 UBC §2211.10.9 Web Stiffener Spacing 333
C709.10 UBC §2211.10.10 Web Stiffener Location................. 333
C709.11 UBC §2211.10.11 Stiffener Welds...................... 333
C709.12 UBC §2211.10.12 Link Beam-Column Connection 333
C709.13 UBC §2211.10.13 Brace and Beam Strength 334
C709.14 UBC §2211.10.14 Column Strength 334
C709.15 UBC §2211.10.15 Roof Link Beam 334
C709.16 UBC §2211.10.16 Concentric Brace in Combination......... 334
C709.17 UBC §2211.10.17 Axial Forces 335
C709.18 UBC §2211.10.18 Beam Flanges 335
C709.19 UBC §2211.10.19 Beam-Column Connection.............. 335

C710 UBC §2211.11 Light Framed Wall Systems....................... 336

Chapter 8
Wood 351

C801 UBC §2301 General Requirements 351

C802 Recommended Modifications to 1994 UBC........................ 351

C802.1 Safe Lateral Strength 351
C802.2 UBC §2314.1 Wood Shear Walls and Diaphragms in Seismic Zones 3 and 4............................... 352
C802.2.1–C802.2.3 No Commentary provided. 352
C802.3 Conventional Light Frame Construction.................... 353

C803 UBC §2302 Definitions 354

C804 UBC §1701.5.16 Special Inspection of Wood Construction 355

Chapter 9
Other Materials 361

C901 General Requirements 361

C902 Recommended Modifications to 1994 UBC........................ 361

C903 Other Sheathing.. 361

Appendices

Appendix A
Engineering Implications of the 1994 Northridge Earthquake . . . 363

Appendix B
Conceptual Framework for Performance-Based Seismic Design 391

Appendix C
SEAOC Strength Design Code Change Proposal 417

List of Tables

Table 104-1 Seismic Zone Factor Z (UBC Table 16-I) 52
Table 104-2 Site Coefficients 52
Table 104-3 Occupancy Categories (similar to UBC Table 16-K) 53
Table 104-4 Vertical Structural Irregularities (UBC Table 16-L) 54
Table 104-5 Plan Structural Irregularities (UBC Table 16-M) 54
Table 104-6 Structural Systems (UBC Table 16-N) 55
Table 104-7 Horizontal Force Factor Cp (UBC Table 16-O) 57
Table 104-8 Factors for Nonbuilding Structures (UBC Table 16-P) 59
Table 152-1 Structural Systems Above the Isolation Interface 60
Table 152-2 Maximum Capable Earthquake Response Coefficient, MM 61
Table 152-3 Near-Field Coefficient, N 62
Table 154-1 Damping Coefficient, B 62
Table 154-2 Site Coefficients for Seismic-Isolated Structures 62
Table 802-1 Maximum Diaphragm Dimension Ratios (UBC Table 23-I-I) 87
Table 802-2 Wall Bracing for Seismic Zones 2B, 3, and 4 (UBC Table 23-I-W) 88
Table 802-3 Cripple Wall Bracing 88
Table C154-1 Damping Coefficients 195
Table C155-1 Limits on Key Design Parameters 196
Table App1D-1 Cross-Modal Coefficients for Equal Modal Damping Ratios of 0.05 for all Modes 230
Table C606-1 Behavior and Limit States—Ductile Material 300
Table C703-1 Summary Statistics From AISC Yield Strengths Survey [AISI, 1994] 342
Table AppA-1 1994 Northridge Earthquake Ground Motion Recorded Near-Fault Rupture. . 386
Table AppB-1 General Damage Descriptions by Performance Levels and Systems 404
Table AppB-2 Performance Levels and Permissible Structural Damage — Vertical Elements 406
Table AppB-3 Performance Levels and Permissible Structural Damage — Horizontal Elements 410
Table AppB-4 Performance Levels and Permissible Damage — Architectural Elements . . . 411
Table AppB-5 Performance Levels and Permissible Damage — Mechanical/Electrical/Plumbing Systems 412
Table AppB-6 Performance Levels and Permissible Damage — Contents 413
Table 16-I Seismic Zone Factor Z 463

Table 16-J Soil Profile Types 463

Table 16-L Vertical Structural Irregularities 464

Table 16-M Plan Structural Irregularities 465

Table 16-N Structural Systems1 466

Table 16-0 Horizontal Force Factors, ap and Rp 469

Table 16-P Ro and Rd Factors for Nonbuilding Structures 473

Table 16-Q Seismic Coefficient Ca 474

Table 16-R Seismic Coefficient Cv 474

Table 16-S Near Source Factor Na1 475

Table 16-T Near Source Factor Nv1 475

Table 16-U Seismic Source Type1 476

Table AppC-1 Pre-Northridge Earthquake Records Close to Fault Rupture 495

Table AppC-2 1994 Northridge Earthquake Records (Sites <= 15 km of Fault Rupture) 496

List of Figures

Figure 104-1 Seismic zone map for California 63
Figure 104-2 Normalized response spectra shapes 64
Figure C104-1 Schematic representation showing how effective peak acceleration and effective peak velocity are obtained from a response spectrum 196
Figure C104-2 Irregularities in elevation 197
Figure C104-4 Irregularities in force transfer 199
Figure C105-1 P-delta symbols and notations. 200
Figure C109-1 Nonbuilding structures supported above grade (WE < 0.25Ws). 201
Figure C109-2 Nonbuilding structures supported above grade (WE< 0.25Ws) 202
Figure C151-1 Isolation system terminology 203
Figure C151-2 Idealized isolation system force-displacement.relationships 204
Figure C151-3 Displacement terminology 205
Figure App1A-1 Horizontal pseudoacceleration response (g) for 1.0 second period and 5 percent damping recorded at soil sites in the 1979 Imperial Valley earthquake 210
Figure App1A-2 Horizontal pseudoacceleration response (g) for 1.0 second periods and 5 percent damping for a magnitude 7.5 earthquake at a soil site according to four widely used prediction equations. 211
Figure App1B-1 Representative response spectrum for soft clay site; damping=5 percent; Z=0.40 214
Figure App1C-1 Resistance versus demand 220
Figure App1D-1 Effect of damping and period ratio on cross-modal coefficient rij. 230
Figure App1E-1 Deformation compatibility consideration of diaphragm deformations. 241
Figure App1E-2 Deformation compatibility consideration of foundation flexibility 242
Figure C407-1 Wall moment versus displacement 285
Figure C407-2 Moment curvature diagrams 286
Figure C407-3 Strain compatibility diagrams 287
Figure C407-4 Approximate determination of øt 288
Figure C606-1 Compressive stress strain diagram for masonry 300
Figure C706-1 Stiffener and doubler plates 343
Figure C706-2 Continuity plate detail. 344
Figure C707-1 Examples of concentric bracing configurations 345
Figure C707-2 Examples of eccentric bracing configurations 345

Figure C707-3 Cyclic response of an axially loaded element [Zayas et al.] 346
Figure C707-4 Chevron bracing in postbuckling stage 346
Figure C707-5 Story shear/story drift diagram for a frame structure with chevron bracing . . 347
Figure C708-1 Design forces for stitches in out-of-plane buckling bracings 347
Figure C708-2 Plastic hinge and free length of gusset plate 348
Figure C709-1 Eccentrically braced frames; examples of deformed geometry showing relationships 348
Figure 709-2 Link beam and intermediate stiffeners 349
Figure C802-1 Subdiaphragm 356
Figure C802-2 Earthquake forces—regular diaphragm 357
Figure C802-3 Earthquake forces—cantilevered diaphragm 357
Figure C802-4 Earthquake forces—building with rotation 357
Figure C802-5 Shear transfer utilizing slant/toenails 358
Figure C802-6 Shear transfer utilizing face nailing at blocks 358
Figure C802-7 Shear transfer utilizing framing anchors 358
Figure C802-8 Height of vertical diaphragm 359
Figure C802-9 Height of vertical diaphragm with openings 360
Figure AppA-1 Comparison of selected spectra from the Northridge earthquake 388
Figure AppA-2 Composite spectra from the Northridge earthquake 388
Figure AppA-3 Ground motion attenuation from the Northridge earthquake 389
Figure AppB-1 Methodology for performance-based engineering 414
Figure AppB-2 Spectrum of seismic damage states 415
Figure AppB-3 Recommended seismic performance objectives for buildings 416
Figure 16-3 Relationship between base shear and displacement 476
Figure 16-4 Design response spectra 477
Figure AppC-1 Determination of distance from nonvertical fault for use in establishing near source factor 502

Preface

This 1996 sixth edition of the *Recommended Lateral Force Requirements and Commentary,* also known as the SEAOC Blue Book, summarizes recommendations of the Structural Engineers Association of California's (SEAOC) for earthquake-resistant design of structures. It represents over six years of work by the SEAOC Seismology Committee.

This 1996 edition of the Blue Book is issued in an entirely new format that is easier to read and reference. Significant format and presentation changes were made so that the Requirements and Commentary would better correlate to each other and with the corresponding sections of the UBC. These major changes necessitated issuing the 1996 edition as a new document, rather than as a series of revision pages to the 1990 edition.

Changes in this Edition of the Blue Book

Reference Document

The 1990 Blue Book referenced three codes: the 1985 UBC, the American Concrete Institute ACI-318-83 requirements, and the American Institute of Steel Construction (AISC) Specification, Eighth Edition. The 1996 Blue Book references only the 1994 UBC.

New Format

Chapters in the 1996 edition parallel those of the 1994 UBC. There are now nine chapters. Listed below are the Blue Book chapters titles and corresponding 1994 UBC chapter titles.

1996 Blue Book Recommendations		1994 UBC	
Chapter 1	General Requirements	Chapter 16	Structural Loads
Chapter 1	§§150 et seq., Seismic Isolated Structures	Chapter 16	Division III, Seismic Isolated Structures
Chapter 2	Structural Tests and Inspection	Chapter 17	Structural Tests and Inspection
Chapter 3	Foundations	Chapter 18	Foundations and Retaining Walls

1996 Blue Book Recommendations	
Chapter 4	Concrete
Chapter 5	Lightweight Metals
Chapter 6	Masonry
Chapter 7	Steel
Chapter 8	Wood
Chapter 9	Other Materials

1994 UBC	
Chapter 19	Concrete
Chapter 20	Lightweight Metals
Chapter 21	Masonry
Chapter 22	Steel
Chapter 23	Wood

UBC Numbering System

The 1996 edition is numbered to make it easier to refer between the 1994 UBC and the Blue Book. The corresponding UBC section number is given at the beginning of subsections.

Chapter 1 - General Requirements

Chapter 1 is similar to the 1990 edition of the Blue Book. The Requirements contain the same complete description of SEAOC-recommended general seismic requirements as did the 1990 document. The Requirements of Chapter 1 are, for the most part, repeated in Chapter 16 of the 1994 UBC. A new section on Existing Structures has been added to this edition of the Blue Book, and that section is consistent with the 1994 UBC. Requirements and Commentary for seismic isolated structures have also been added.

Materials Chapters 4, 5, 6, 7, and 8

These chapters reference the 1994 UBC. Included in the Requirements are only those SEAOC recommendations—additions or deletions—not found in the 1994 UBC. For example, the steel chapter no longer explicitly gives requirements for eccentric braced frames (EBF) because these are in the 1994 UBC. The material chapters are short and do not repeat design provisions given in the 1994 UBC.

Commentary

Commentary chapters include material from the 1990 Recommendations plus corrections, discussions of revisions, and additions corresponding to new code changes, and enhancements. The commentary covers all 1994 UBC sections pertinent to seismic design.

Engineering Implications of the 1994 Northridge Earthquake

Commentary has been added as Appendix A on the engineering implications of the 1994 Northridge earthquake.

Conceptual Framework for Performance-Based Seismic Design

Commentary from the SEAOC Vision 2000 Committee has been added as Appendix B, addressing development of performance-based design criteria.

SEAOC Strength Design Code Change Proposal

These new strength design recommendations have been developed by the Seismology Committee and its Ad Hoc Committee. The recommendations were approved by the ICBO Lateral Force Design Committee at its September 1996 meeting, and are presented as Appendix C.

Summary of Significant Technical Changes

Clarifications and new Requirements have been added to this edition of the Blue Book. Major changes are summarized below. Many other minor changes, not described below, have also been incorporated into these Recommendations.

1. ***Building Separations***

   Smaller separations are permitted under the new exception to the present $3R_w/8$ times the displacement due to seismic forces criteria.

2. ***Strength Ratio***

   An alternate equation is added to relax the restrictive requirements for the column-to-beam-strength ratio.

3. ***Boundary Members for Reinforced Concrete Shear Walls***

   This new approach supersedes previous criteria for boundary member design. The new approach is based on more rational procedures and results in reduced overcrowding of reinforcement in boundary members. The reduction of required flexural reinforcement in the boundary zone improves the possibility of a more ductile flexural yielding mode, precluding the less ductile shear failure mode.

4. ***New Requirements for the Strength Design of Masonry Structures***

   Requirements have been added for:

   - Beams, Piers, and Columns
   - Moment-Resisting Wall Frames
   - Wall Design for Out-of-Plane Loads Greater than $0.04 f'_m$
   - Reinforcement for Strength Design.

   These Requirements provide an alternative method to the working stress design method.

5. ***Flexible Diaphragms***

   Design forces for flexible diaphragms and their connections that provide lateral support for walls or frames of masonry or concrete shall be calculated using an R_w not to exceed 6.

6. ***Special Concentrically Braced Steel Frames with SMRF***

   New Requirements have been added that improve local buckling behavior.

7. ***Ordinary Concentrically Braced Steel Frames***

   Width-to-thickness requirements have been changed to conform to those for Special Concentrically Braced Frames.

8. ***Cantilevered Column***

   A definition and a value for R_w for the cantilevered element and system have been added.

9. ***Deformation Compatibility***

   Clarification has been added regarding the lateral deformation of the gravity load-carrying members and their connections.

10. ***Anchorage of Concrete and Masonry Walls***

    Requirements have been modified for concrete and masonry wall anchorages to floors and roofs.

11. ***Lateral Reinforcement for Concrete Columns***

    Confinement reinforcement has been added for reinforced concrete columns that are not part of the lateral force-resisting system.

12. ***Plywood Shear Wall Aspect Ratio***

    The maximum aspect ratio for plywood shear walls has been changed from 3-1/2:1 to 2:1.

13. ***Welded Steel Joints in Special Moment Resisting Frames***

    Prescriptive welded steel joint design requirements in SMRFs have been deleted.

14. ***Subdiaphragm Aspect Ratio***

    The maximum aspect ratio for plywood subdiaphragms has been changed from 4:1 to 2-1/2:1

15. ***Seismic-Isolated Structures***

    Requirements for seismic-isolated structures have been updated to reflect improvements in design criteria. New commentary has been added to Chapter 1.

16. ***Liquefaction Potential and Soil Strength Loss***

    A requirement for investigation of liquefaction potential and soil strength loss has been added.

Acknowledgments

The changes in this 1996 edition of the *Recommended Lateral Force Requirements and Commentary* reflect the work of the 1989-90 through 1995-96 SEAOC Seismology Committees. Members of these committees are listed below. This 1996 edition of the Recommendations has built upon the work of all the SEAOC Seismology Committees over a 38-year period. A list of all past and present members of the Committee is included.

The actual task of developing the 1996 Recommendations was performed by the 1996 Blue Book Committee, which proved to be a major task. Members of the Blue Book Committee who contributed substantial work on the document are listed below.

1996 Blue Book Committee

- Ronald P. Gallagher (SEAONC)
  - Chairman
- Theodore C. Zsutty (SEAONC)
  - Co-Chairman
- Robert Bachman (SEAOSC)
- Vera Berger (SEAOSD)
- Steven T. Brokken (SEAONC)
  - Lead Chapter 6
- Sigmund A. Freeman (SEAONC)
- Robert N. Chittenden (SEAOCC)
  - Lead Chapter 1
- Philip G. Griffin (SEAONC)
- Tom H. Hale (SEAOCC)
- Ronald O. Hamburger (SEAONC)
- Edward A. Hanlon (SEAOSD)
- Saif M. Hussain (SEAOSC)
- Mark S. Jokerst (SEAONC)
  - Lead Chapter 4
- Charles A. Kircher (SEAONC)
  - Lead Seismic Isolation
- Marshall Lew (SEAOSC)
- James O. Malley (SEAONC)
  - Lead Chapter 7
- Frank E. McClure (SEAONC)
- David L. McCormick (SEAONC)
- Daniel McNaughton (SEAOSD)
  - Lead Chapters 5, 8, and 9
- Andrew T. Merovich (SEAONC)
- Vilas S. Mujumdar (SEAOSD)
- Farzad Naeim (SEAOSC)
- Gerald B. Neville (SEAOSC)
- Richard J. Phillips (SEAOSC)
- Maurice S. Power (SEAONC)
- Edward Rinne (SEAONC)
  - Lead Chapter 3
- Ali M. Sadre (SEAOSD)
- C. Mark Saunders (SEAONC)
- Scott A. Stedman (SEAOSD)
- Ian R. Stubbs (SEAOSC)
- Robert C. Thacker (SEAOCC)
- Charles C. Thiel Jr. (SEAONC)
- Ajit S. Virdee (SEAOCC)
  - Lead Preface
- Andrew S. Whittaker (SEAONC)
- Nabih F. G. Youssef (SEAOSC)
- Edwin G. Zacher (SEAONC)
  - Lead Chapter 2

1995-1996 Seismology Committee

David Bonneville (SEAONC)
Chairman
Robert Bachman (SEAOSC)
Past Chairman
Ajit S. Virdee (SEAOCC)
Tom H. Hale (SEAOCC)
Saif M. Hussain (SEAOSC)
Richard Phillips (SEAOSC)
Thomas A. Sabol (SEAOSC)
Dominic E. Campi (SEAONC)
Roumen Mladjov (SEAONC)
Mark S. Jokerst (SEAONC)
Ali M. Sadre (SEAOSD)
Carl B. Schulze (SEAOSD)
Stacy Bartoletti (SEAONC)
Assistant to the Chairman

1994-1995 Seismology Committee

Robert E. Bachman (SEAOSC)
Chairman
Robert N. Chittenden (SEAOCC)
Past Chairman
Ajit S. Virdee (SEAOCC)
Tom H. Hale (SEAOCC)
Saif M. Hussain (SEAOSC)
Nabih F. G. Youssef (SEAOSC)
Thomas A. Sabol (SEAOSC)
David R. Bonneville (SEAONC)
Mark S. Jokerst (SEAONC)
Dominic E. Campi (SEAONC)
Ali M. Sadre (SEAOSD)
Carl B. Schulze (SEAOSD)
Kamy Rouintan (SEAOSC)
Assistant to the Chairman

1993-1994 Seismology Committee

Robert N. Chittenden (SEAOCC)
Chairman
Vilas S. Mujumdar (SEAOSD)
Past Chairman
Thomas H. Hale (SEAOCC)
Ajit S. Virdee (SEAOCC)
Robert E. Bachman (SEAOSC)
Nabih F. G. Youssef (SEAOSC)
Thomas A. Sabol (SEAOSC)
Charles A. Kircher (SEAONC)
David R. Bonneville (SEAONC)
Mark S. Jokerst (SEAONC)
Daniel McNaughton (SEAOSD)
Ali M. Sadre (SEAOSD)

1992-1993 Seismology Committee

Vilas S. Mujumdar (SEAOSD)
Chairman
Ronald P. Gallagher (SEAONC)
Past Chairman
Edward A. Hanlon (SEAOSD)
Daniel McNaughton (SEAOSD)
Robert N. Chittenden (SEAOCC)
Ronald O. Hamburger (SEAONC)
Ajit S. Virdee (SEAOCC)
Nabih F. G. Youssef (SEAOSC)
Robert Bachman (SEAOSC)
Richard J. Phillips (SEAOSC)
Charles A. Kircher (SEAONC)
David R. Bonneville (SEAONC)

1991-1992 Seismology Committee

Ronald P. Gallagher (SEAONC)
Chairman
Nabih F. G. Youssef (SEAOSC)
Past Chairman
Charles A. Kircher (SEAONC)
Ronald O. Hamburger (SEAONC)
Andrew T. Merovich (SEAONC)
Edward A. Hanlon (SEAOSD)
Vilas S. Mujumdar (SEAOSD)
Robert N. Chittenden (SEAOCC)
Ajit S. Virdee (SEAOCC)
Richard J. Phillips (SEAOSC)
Robert Bachman (SEAOSC)
Thomas A. Sabol (SEAOSC)

1990-1991 Seismology Committee

Nabih F. G. Youssef (SEAOSC)
Chairman
Eugene E. Cole (SEAOCC)
Past Chairman
Robert Bachman (SEAOSC)
Richard J. Phillips (SEAOSC)
Thomas A. Sabol (SEAOSC)
Andrew T. Merovich (SEAONC)
Ronald O. Hamburger (SEAONC)
Ronald P. Gallagher (SEAONC)
Edward A. Hanlon (SEAOSD)
Robert O. Hendershot (SEAOSD)
Robert N. Chittenden (SEAOCC)
Ajit S. Virdee (SEAOCC)

1989-1990 Seismology Committee

Eugene E. Cole (SEAOCC)
Chairman
James S. Lai (SEAOSC)
Past Chairman
Robert C. Thacker (SEAOCC)
Robert N. Chittenden (SEAOCC)
Ajit S. Virdee (SEAOCC)
Robert Bachman (SEAOSC)
Nabih F. G. Youssef (SEAOSC)
Gary C. Hart (SEAOSC)
Andrew T. Merovich (SEAONC)
Ronald P. Gallagher (SEAONC)
C. Mark Saunders (SEAONC)
Edward A. Hanlon (SEAOSD)
Robert O. Hendershot (SEAOSD)†
Vilas S. Mujumdar (SEAOSD)†

† Alternate

SEAOC Seismology Committee Members 1958-1996

Bachman, Robert (SEAOSC)
1989-1994, 1995,* 1996
Barnes, S. B. (SEAOSC)
1959, 1960
Barrish, Jack S. (SEAOCC)
1960, 1974-1976, 1976,* 1979
Binder, R. W. (SEAOSC)
1959, 1960
Blume, John A. (SEAONC)
1958-1960
Bonneville, David R. (SEAONC)
1993-1996
Brandley, R. W. (SEAOCC)
1960
Brandow, Gregg (SEAOSC)
1985-1987
Brownfield, Allen H. (SEAOCC)
1960
Brugger, Walter A. (SEAOSC)
1965
Buehler, Walter D. (SEAOCC)
1968
Byrum, Robert C. (SEAOSC)
1968
Campbell, David B. (SEAOCC)
1985
Campi, Dom (SEAONC)
1995-1996
Chittenden, Robert (SEAOCC)
1990-1995, 1993,* 1994
Christensen, Ted (SEAOSC)
1982-1984
Clough, Raymond W. (SEAONC)
1966, 1967
Cole, Eugene E. (SEAOCC)
1979-1984, 1989,* 1990
Crandall, L. Leroy (SEAOSC)
1960
Crane, David A. (SEAOCC)
1965-1968
Degenkolb, Henry J. (SEAONC)
1959, 1960
Dixon, Schaefer J. (SEAOSC)
1975, 1976
Doig, Bruce (SEAOCC)
1987, 1988
Donovan, Neville (SEAONC)
1975, 1976
Driskell, John J. (SEAOSC)
1965-1968
Elsesser, Eric (SEAONC)
1974-1976
Erick, Murray (SEAOSC)
1959, 1960
Ewing, Merle A. (SEAONC)
1960
Finch, Herman F. (SEAOCC)
1959, 1960
Flores, Raymond (SEAOSD)
1981-1983
Forell, Nicholas (SEAONC)
1980-1982
Foth, Ullrich A. (SEAOSC)
1975, 1976, 1987
Fratessa, Paul F. (SEAONC)
1981-1985, 1984,* 1985
Freeman, Sigmund A. (SEAONC)
1975-1987
Gallagher, Ronald P. (SEAONC)
1989-1991, 1992,* 1993
Gates, William R. (SEAOSC)
1979-1981
Gilligan, Mark (SEAONC)
1985-1987
Gordon, Ben (SEAOSC)
1968, 1969
Graham, Leslie W. (SEAONC)
1968
Green, Norman B. (SEAOSC)
1968
Greenlaw, Charles (SEAOCC)
1985
Greve, Norman R. (SEAOSC)
1975-1978
Griffin, Phillip G. (SEAONC)
1984-1986
Hale, Tom H. (SEAOCC)
1994-1996

*Chairman

Hamburger, Ronald (SEAONC)
1991-1993
Hanlon, Jr., Edward A. (SEAOSD)
1988, 1993
Hammill, Harold B. (SEAONC)
1959, 1960
Hart, Gary C. (SEAOSC)
1988
Hart, Gordon M. (SEAOCC)
1960
Hendershot, Robert (SEAOSD)
1991, 1992
Holmes, William T. (SEAONC)
1977-1979
Hussain, Saif (SEAOSC)
1995-1996
Johnson, Albin W. (SEAOSC)
1985*
Johnson, Edwin H. (SEAOSD)
1972-1977, 1983-1986
Johnson, James (SEAOSC)
1977-1979
Johnston, Roy G. (SEAOSC)
1959, 1960, 1969-1971
Jokerst, Mark (SEAONC)
1993-1996
Jones, Norman (SEAOSC)
1967
Kamei, Tom T. (SEAOSC)
1971, 1973
Kariotis, John C. (SEAOSC)
1974-1977, 1979
Kellam, H. S. (Pete) (SEAONC)
1959, 1960, 1965-1970, 1968*
Kesler, James J. (SEAOSC)
1966-1968
Kircher, Charles (SEAONC)
1992-1994
Knox, M. T. (SEAOCC)
1958, 1960
Lai, James S. (SEAOSC)
1986-1988, 1988,* 1989
Larson, Marvin A. (SEAONC)
1968, 1969
Lehmer, Gerald D. (SEAOSC)
1978-1985
Libby, James R. (SEAOSD)
1978-1982, 1981,* 1982
Lindskog, Ed (SEAOSC)
1958, 1960
Luttrell, Kenneth (SEAOCC)
1983-1987, 1983*Lyons, Keven P.
(SEAOSD)
1988
Maag, Ernst (SEAOSC)
1960
Magee, Douglas H. (SEAOSD)
1984-1986
Manning, Joseph G. (SEAONC)
1975, 1976
Mark, Melvin H. (SEAOSD)
1970-1976, 1977*
Mathiesen, R. B. (Fritz) (SEAONC)
1979-1981
McClure, Frank E. (SEAONC)
1960
McNaughton, Daniel L. (SEAOSD)
1977-1981, 1993
Meehan, John F. (SEAOCC)
1959, 1960, 1969-1974
Merovich, Andrew (SEAONC)
1990-1992
Messinger, David L. (SEAONC)
1982-1984
Mladjov, Roumen (SEAONC)
1995-1996
Moore, William W. (SEAONC)
1960
Mujumdar, Vilas S. (SEAOSD)
1991-1993, 1993*
Nicoletti, Joseph P. (SEAONC)
1970-1973
Omsted, Herald (SEAOSC)
1959, 1960
Parme, Alfred L. (SEAOSD)
1984
Phillips, Richard (SEAOSC)
1991-1993, 1996
Pinkham, Clarkson W. (SEAOSC)
1965-1970, 1972-1975, 1967-1970*
Porush, Allan (SEAOSC)
1982-1988, 1987,* 1988*

*Chairman

Powers, Henry C. (SEAONC)
1959, 1960
Preece, F. Robert (SEAONC)
1960, 1983-1985
Ragsdale, John T. (SEAOCC)
1977
Richter, Phillips J. (SEAOSC)
1980-1984, 1982,* 1983,* 1984, 1987
Rinne, John E. (SEAONC)
1959, 1960 1965-1967
Robb, John O. (SEAOSC)
1977-1979, 1979,* 1980
Rumberger, William D. (SEAOCC)
1960, 1967-1969, 1975, 1976, 1980-1982
Sabol, Thomas A. (SEAOSC)
1994-1996
Sadre, Ali M. (SEAOSD)
1994-1996
Sauer, Arthur A. (SEAOCC)
1960, 1966
Saunders, C. Mark (SEAONC)
1987-1989
Schulze, Carl (SEAOSD)
1994-1996
Sedgewick, G.A. (SEAONC)
1959, 1960
Seed, H. Bolton (SEAONC)
1974-1976
Sharpe, Roland L. (SEAONC)
1969, 1972,* 1973,* 1974
Skinner, M. J. (SEAOSC)
1960
Smith, W. Hudson (SEAONC)
1975-1979
Steinbrugge, John M. (SEAOSC)
1960
Steinman, Hans G. (SEAOSC)
1971,* 1972
Strand, Donald R. (SEAOSC)
1971-1975, 1975*
Tandowsky, Sanford (SEAONC)
1970-1974
Teal, Edward J. (SEAOSC)
1970, 1982*
Teixeira, Stanley E. (SEAONC)
1965, 1966
Thacker, Robert C. (SEAOCC)
1986-1990
Thiel, Charles C. (SEAONC)
1987
Thompson, James H. (SEAOSC)
1970
Travis, William L. (SEAOSC)
1969, 1971, 1972, 1982, 1983
Venolia, Kenneth V. (SEAOCC)
1965-1967
Virdee, Ajit S. (SEAOCC)
1972-1976, 1974,* 1975, 1988-1996
Vossenkemper, Earl H. (SEAOSC)
1969, 1970
Wheeler, William T. (SEAOSC)
1959,* 1960*
Whitelaw, Robert (SEAOSC)
1985-1987
Wilder, Robert (SEAOSC)
1960
Willsea, Frederick J. (SEAONC)
1988-1990
Wong, Hoi W. (SEAOCC)
1978, 1979
Wosser, Thomas D. (SEAONC)
1971-1974
Wylie, Loring A. (SEAONC)
1975-1977
Youssef, Nabih F.G. (SEAOSC)
1988, 1990-1995, 1991*
Zacher, Edwin G. (SEAONC)
1972-1977, 1976,* 1977, 1986-1988, 1988*
Zsutty, Theodore (SEAONC)
1978-1980,* 1981

*Chairman

Suggestions for Improvement

In keeping with its charge of maintaining a "living" document and keeping abreast of the state of the art, the SEAOC Seismology Committee plans to update these Recommendations as new research and better understanding of building earthquake performance becomes available. Comments and suggestions for improvement are welcome and should be addressed to:

Chair, SEAOC Seismology Committee
555 University Avenue, Suite 126
Sacramento, California 95825-6510
Telephone: (916) 427-3647
Fax: (916) 568-0677

Reference to 1994 UBC

The following sections of the 1994 UBC are referenced in this edition of the Blue Book.

	1996 SEAOC Blue Book	
UBC §	**Reqs**	**Commentary**
1624	101.1-5	C101.1-5
1625	102	C102
1626	103	C103
1627	104.1-10	C104.1-11
1628	105.1-10	C105.1-10
1629	106.1-5	C106.1-5
1630	107	C107
1630.1	107.1	C107.1
1630.2	107.2	C107.2
1630.3	107.3	C107.3
1630.4	107.4	C107.4
1630.5	107.5	C107.5
1631	108	C108
1631.2.8	108.2.8	
1632	109.1-5	C109.1-5
1650	150	C150
1651	151	
1652	152	
1653	153.1-5	C153.1-5
1654	154.1-6	C154.2-6
1655	155.1-8	C155.1-7
1656	156.1-2	C156.1-2
1657	157.1-3	C157.3
1658	158	
1659	159	
1660	160.1-2	
1661	161	
1701	202.1	C202.1
1702	202.2	C202.2
1705	202.3	C202.3
1706	202.4	C202.4
1707	202.5	C202.5
1805	302.1	C302.1
1809	302.2	C302.2

	1996 SEAOC Blue Book	
UBC §	Reqs	Commentary
1809.5.1		C302.1
1907	402.1	C402.1
1909.3.4	402.2	C402.2
1910.8.4	402.3	C402.3
1911.5.6.9	402.4	C402.4
1921	402.5	
1921.2.3		C403.1
1921.2.4.2		C403.2
1921.2.5		C403.3
1921.3	402.6	
1921.3.1		C404.1
1921.3.2		C404.2
1921.3.3		C404.3
1921.3.3.4		C404.4
1921.3.4.1		C404.5
1921.4	402.7	C405
1921.4.1		C405.1
1921.4.2		C405.2
1921.4.3.2		C405.3
1921.4.4		C405.4
1921.4.5.1		C405.5
1921.5		C406
1921.5.1		C406.1
1921.5.3	402.8	
1921.5.3.1		C406.2
1921.5.4.1		C406.3
1921.6.2		C407.1
1921.6.5		C407.2
1921.6.5	402.9	
1921.6.6.3	402.1	C407.3
1921.6.8		C407.4

	1996 SEAOC Blue Book	
UBC §	Reqs	Commentary
1921.6.9		C407.5
1921.6.9.1	402.11	
1921.6.10		C407.6
1921.6.11	402.12	C407.7
1921.6.12		C407.8
1921.7	402.13	
1921.7.2		C408.1
1921.7.3		C408.2
2106.1.12.4		C604.1
2106.2.14		C604.2
2108.1		C606.1
2108.1.2		C606.2
2108.1.4		C606.3
2108.2	602.4	
2211.1		C709
2211.2		C701.1
2311.3.1		C802.2
2211.4.1		C703.1
2211.4.2		C703.2
2211.5		C704
2211.5.1		C704.1
2211.5.2		C704.2
2211.5.3		C704.3
2211.7.1		C706.1
2211.7.1.3	702.1	
2211.7.1.3	702.2	
2211.7.2		C706.2
2211.7.3		C706.3
2211.7.4		C706.4
2211.7.5		C706.5
2211.7.6		C706.6

	1996 SEAOC Blue Book	
UBC §	**Reqs**	**Commentary**
2211.7.7		C706.7
2211.7.8		C706.8
2211.7.9		C706.9
2211.7.10		C706.10
2211.8.1		C707.1
2211.8.2		C707.2
2211.8.3		C707.3
2211.8.4		C707.4
2211.8.5		C707.5
2211.8		C707
2211.9		C708
2211.9.1		C708.1
2211.9.2		C708.2
2211.9.3		C708.3
2211.9.4		C708.4
2211.9.5		C708.5
2211.9.6		C708.6
2211.10.1		C709.1
2211.10.2		C709.2
2211.10.3		C709.3
2211.10.4		C709.4
2211.10.5		C709.5
2211.10.6		C709.6
2211.10.7		C709.7
2211.10.8		C709.8
2211.10.9		C709.9
2211.10.10		C709.10
2211.10.11		C709.11
2211.10.12		C709.12
2211.10.13		C709.13
2211.10.14		C709.14

	1996 SEAOC Blue Book	
UBC §	**Reqs**	**Commentary**
2211.10.15		C709.15
2211.10.16		C709.16
2211.10.17		C709.17
2211.10.18		C709.18
2211.10.19		C709.19
2211.11.11		C710
2301		C801
2302	802.1	C802.1
2314		C802.3
2314.5	802.3	
2326		C802.4
3400	110	
3401	110.1	C110.1
3402	110.2	
3403	110.3	
3404	110.4	
3405	110.5	

CHAPTER 1

General Requirements for Design and Construction of Earthquake-Resistive Structures

Note: Wording that is substantially different from that found in the UBC is marked with a changebar in the margin.

101 UBC §1624 General

101.1 Scope and Objectives

Every structure and every portion thereof shall, as a minimum, be designed, detailed, and constructed to resist the effects of seismic ground motions as provided in these Requirements. Any jurisdiction may adopt more stringent requirements.

The purpose of the earthquake design Requirements herein is primarily to safeguard against major structural failures and loss of life, not to limit damage or maintain function.

101.2 Wind and Seismic Forces

Where code prescribed wind design produces greater effects the wind design shall govern, but detailing requirements and limitations prescribed in these provisions for seismic design shall be followed.

101.3 Continuous Load Path

A continuous load path, or paths, with adequate strength and stiffness shall be provided to transfer all forces from the point of application to the resisting elements.

101.4 Basis for Seismic Design

The basis for seismic design shall be stated on the structural drawings. As a minimum, the statement shall include:

1. A statement of the governing edition of the building code.
2. Total base shear coefficient used for seismic design.

3. Description of the lateral force resisting system, as defined in these Requirements.

101.5 Computer Calculations

Calculations may include the results from an electronic digital computer program. The following Requirements apply to calculations submitted to a building official that include such computer output.

101.5.1 A drawing of the complete mathematical model used to represent the structure in the computer-generated analysis shall be provided.

101.5.2 A program description (User's Guide) shall be available and contain the information necessary to determine the nature and extent of the analysis, verify the input data, interpret the results, and determine whether the computations comply with these Requirements.

101.5.3 Data provided as computer input shall be clearly distinguished from those computed in the program. The information required in the output shall include date of processing, program identification, identification of structures being analyzed, all input data, units, and final results.

101.5.4 The first sheet of each computer run shall be signed by the engineer responsible for the structural design.

102 UBC §1625 Definitions

Base. The level at which the earthquake motions are considered to be imparted to the structure, or the level at which the structure as a dynamic vibrator is supported.

Base Shear (V). *V* is the total design lateral force or shear at the base of a structure.

Bearing Wall System. A structural system without a complete vertical load-carrying frame. See Section 104.6.1.

Boundary Element. An element at edges of openings or at perimeters of shear walls or diaphragms.

Braced Frame. An essentially vertical truss system of the concentric or eccentric type provided to resist lateral forces.

Building Frame System. An essentially complete frame that provides support for gravity loads. See Section 104.6.2.

Cantilevered Column Element. A column element of a lateral force resisting system that cantilevers from a fixed base and is hinged at the top with lateral forces applied essentially at the hinge.

Collector. A member or element provided to transfer lateral forces from a portion of a structure to vertical elements of the lateral force resisting system.

Concentric Braced Frame. A braced frame in which the members are subjected primarily to axial forces.

Diaphragm. A horizontal or nearly horizontal system acting to transmit lateral forces to the vertical resisting elements. The term "diaphragm" includes horizontal bracing systems.

Diaphragm Chord. The boundary element of a diaphragm or shear wall that is assumed to take axial stresses in a manner analogous to the flanges of a beam.

Diaphragm Strut (drag strut, tie, collector). The element of a diaphragm parallel to the applied load that collects and transfers diaphragm shear to the vertical resisting elements or distributes loads within the diaphragm. Such members may take axial tension or compression.

Drift. See Story Drift.

Dual System. A combination of moment resisting frames and shear walls or braced frames designed in accordance with the criteria of Section 104.6.4.

Eccentric Braced Frame (EBF). A steel braced frame designed in conformance with Chapter 7, Structural Steel.

Essential Facilities. Those structures necessary for postearthquake emergency operations.

Flexible Element or System. An element or system whose deformation under lateral load is significantly larger than adjoining parts of the system. Limiting ratios for defining specific flexible elements are set forth in Section 105.5.1 or 107.2.2.

Horizontal Bracing System. A horizontal truss system that serves the same function as a diaphragm.

Intermediate Moment Resisting Frame (IMRF). A concrete frame designed in accordance with Chapter 4, Reinforced Concrete.

Lateral Force Resisting System. That part of the structural system assigned to resist lateral forces.

Moment Resisting Frame (MRF). A frame in which members and joints are capable of resisting forces primarily by flexure.

Moment Resisting Wall Frame (MRWF). A masonry wall frame designed and detailed in accordance with Chapter 6, Reinforced Masonry.

Ordinary Moment Resisting Frame (OMRF). A moment resisting frame not meeting special detailing requirements for ductile behavior.

Orthogonal Effects. The effects on the structure due to earthquake motions acting in directions other than parallel to the direction of resistance under consideration.

P-delta (Δ) Effect. The secondary effect on shears, axial forces, and moments of frame members induced by the vertical loads acting on the laterally displaced building frame.

Platform. The lower rigid portion of a structure having a vertical combination of structural systems for use in Section 104.8.1.

Shear Wall. A wall designed to resist lateral forces parallel to the plane of the wall (sometimes referred to as a vertical diaphragm or a structural wall).

Soft Story. One in which the lateral stiffness is less than 70 percent of the lateral stiffness of the story immediately above.

Space Frame. A three-dimensional structural system without bearing walls composed of members interconnected so as to function as a complete self-contained unit with or without the aid of horizontal diaphragms or floor bracing systems.

Special Concentrically Braced Frame. A steel braced frame designed in conformance with the Requirements of Chapter 7, Structural Steel.

Special Moment Resisting Frame (SMRF). A moment resisting frame specially detailed to provide ductile behavior and comply with the Requirements given in Chapters 4, Reinforced Concrete, or Chapter 7, Structural Steel.

Story. The space between levels. Story x is the story below level x.

Story Drift. The displacement of one level relative to the level immediately above or below.

Story Drift Ratio. Story drift divided by the story height.

Story Shear, V_x. The summation of design lateral forces above the story under consideration.

Strength. The capacity of an element or a member to resist factored loads, as prescribed in these Recommendations.

Structure. An assemblage of framing members designed to support gravity loads and resist lateral forces. Structures may be categorized as building structures or nonbuilding structures.

Subdiaphragm. A portion of a larger wood diaphragm designed to anchor and transfer local forces to primary diaphragm struts and the main diaphragm.

Tower. The upper flexible portion of a structure having a vertical combination of structural systems for use in Section 104.8.2.

Vertical Load-Carrying Frame. A space frame designed to carry all gravity loads.

Wall Anchorage System. The system of elements anchoring the wall to the diaphragm and those elements within the diaphragm required to develop the anchorage forces, including subdiaphragms and continuous ties, as specified in Sections 108.2.8 and 108.2.9.

Weak Story. One in which the story strength is less than 80 percent of that of the story above. See Table 104-4.

103 UBC §1626 Symbols and Notations

The following symbols and notations apply to the provisions of this section:

A_c = The combined effective area, in square feet, of the shear walls in the first story of the structure, as determined from Equation 105-4.

A_e = The minimum cross-sectional area in any horizontal plane in the first story, in square feet, of a shear wall for use in Equation 105-4.

A_x = The torsional amplification factor at level x, as determined from Equation 105-9.

C = Numerical coefficient specified in Section 105.2.1.

C_f = Numerical coefficient specified in Section 107.2.

C_g = Numerical coefficient specified in Section 107.2.

C_p = Numerical coefficient specified in Section 107 and given in Table 104-7.

C_t = Numerical coefficient given in Section 105.2.2.

D_e = The length, in feet, of a shear wall in the first story in the direction parallel to applied forces.

DL = Dead load on a structural element.

δ_i = Horizontal displacement at Level i relative to the base due to applied lateral forces, f_i, for use in Equation 105-5.

E = Load due to earthquake on a structural element.

f_i = Lateral force at Level i for use in Equation 105-5.

F_i , F_n, F_x= Lateral force applied to Level i, n, or x, respectively.

F_p = Lateral force on a part of the structure or equipment supported by a structure.

F_t = That portion of the base shear, V, considered concentrated at the top of the structure in addition to F_n.

g = Acceleration due to gravity.

h_i, h_n, h_x = Height in feet above the base to Level i, n, or x, respectively.

I = Importance factor given in Table 104-3.

I_p = Importance factor specified for an element, component of structure, or equipment in Section 107.2 and Table 104-3.

Level i = Level of the structure referred to by the subscript i; $i = 1$ designates the first level above the base.

Level n = The uppermost level in the main portion of the structure.

Level x = The level under design consideration: $x = 1$ designates the first level above the base.

LL = Live load on a structural element.

R_w = Numerical coefficient given in Tables 104-6 and 104-8.

S = Site coefficient for soil characteristics given in Table 104-2.

T = Fundamental period of vibration, in seconds, of the structure in the direction under consideration.

V = The total design lateral force or shear at the base.

V_x = The design story shear in story x.

W = The total seismic dead load defined in Section 105.1.3.

w_i, w_x = That portion of W located at or is assigned to Level i or x, respectively.

w_{px} = The weight of the diaphragm and the elements tributary thereto at Level x, including applicable portions of other loads defined in Section 105.1.3.

W_p = The weight of an element, component, or equipment.

Z = Seismic zone factor given in Table 104-1.

104 UBC §1627 Criteria Selection

104.1 Basis for Design

The procedures and limitations for the design of structures shall be determined considering zoning, site characteristics, occupancy, configuration, structural system, and height, in accordance with this section. The minimum design seismic forces shall be those determined in accordance with the static lateral force procedure of Section 105, except as modified by Section 106.4.1.3.

104.2 Seismic Zones

Each site shall be assigned to a seismic zone in accordance with Figure 104-1. Each structure shall be assigned a zone factor, Z, in accordance with Table 104-1.

104.3 Site Geology and Soil Characteristics

Soil profile type and site coefficient, S, shall be established in accordance with Table 104-2.

104.4 Occupancy Categories

For purposes of earthquake resistant design, each structure shall be placed in one of the occupancy categories listed in Table 104-3. Table 104-3 lists importance factors I. Design and construction review requirements for each category are given in Section 201.

104.5 Configuration Requirements

Each structure shall be designated as being structurally regular or irregular.

104.5.1 ***Regular Structures.*** Regular structures have no significant physical discontinuities in plan or vertical configuration or in their lateral force resisting systems such as the irregular features described in Section 104.5.2.

104.5.2 ***Irregular Structures.*** Irregular structures have significant physical discontinuities in plan or vertical configuration or in their lateral force resisting systems. Irregular features include, but are not limited to, those described in Tables 104-4 and 104-5.

1. **Vertical Irregularity.** Structures having one or more of the features listed in Table 104-4 shall be designated as having a vertical irregularity.

   ***Exception*:** Where no story drift ratio under design lateral force is greater than 1.3 times the story drift ratio of the story above, the structure may be deemed to not have irregularities of Types A or B in Table 104-4. The story drift ratio for the top two stories need not be considered. The story drifts for this determination may be calculated neglecting torsional effects.
2. **Plan Irregularity.** Structures having one or more of the features listed in Table 104-5 shall be designated as having a plan irregularity.

104.6 Structural Systems

Structural systems shall be classified as one of the types defined below and listed in Table 104-6.

104.6.1 ***Bearing Wall System.*** A structural system without a complete vertical load-carrying frame. Bearing walls or bracing systems provide support for gravity loads. Resistance to lateral load is provided by shear walls or braced frames.

104.6.2 ***Building Frame System.*** A structural system with an essentially complete frame providing support for gravity loads. Resistance to lateral load is provided by shear walls or braced frames.

For the purpose of determining R_w, laterally braced shear walls as defined in Table 104-6 may support factored gravity loads, including self-weight, not exceeding 10 percent of the nominal axial strength.

104.6.3 ***Moment Resisting Frame System.*** A structural system with an essentially complete frame providing support for gravity loads. Moment resisting frames provide resistance to lateral load primarily by flexural action of members.

104.6.4 ***Dual System.*** A structural system with the following features:

104.6.4.1 An essentially complete frame providing support for gravity loads.

104.6.4.2 Resistance to lateral forces is provided by shear walls or braced frames and a specially detailed moment resisting frame system. The frame shall be designed to independently resist at least 25 percent of the design base shear.

104.6.4.3 The two systems shall be designed to resist the total base shear in proportion to their relative rigidities considering the interaction of the dual system at all levels.

104.6.5 ***Cantilevered Column System.*** A structural system comprised primarily of cantilevered column elements.

104.6.6 ***Undefined Structural System.*** A structural system not listed in Table 104-6.

104.6.7 ***Nonbuilding Structural System.*** A structural system conforming to Section 109.

104.7 Height Limits

Height limits for the various structural systems in Seismic Zones 3 and 4 are given in Table 104-6.

> *Exception*: Under the following conditions, regular structures may exceed these limits by not more than 50 percent:
>
> 1. Unoccupied structures not accessible to the general public.
> 2. Other structures, when technical data is submitted per Section 104.10 and the special review Requirements of Section 201 are satisfied.

104.8 Selection of Lateral Force Procedure

104.8.1 ***General.*** Any structure may be, and certain structures defined below shall be, designed using the dynamic lateral force procedures of Section 106.

104.8.2 ***Static Procedure.*** The static lateral force procedure of Section 105.2 may be used for the following structures:

1. All structures, regular or irregular, in Seismic Zone 1 and in Occupancy Category IV in Seismic Zone 2.
2. Regular structures under 240 feet in height with lateral force resistance provided by systems listed in Table 104-6 except where Section 104.8.3, Item 4 applies.
3. Irregular structures not more than five stories or 65 feet in height.
4. Structures having a tower supported on a platform conforming to the following:
   - a. Both portions of the structure, when considered separately, can be classified as regular.
   - b. The average story stiffness of the platform is at least 10 times the average story stiffness of the tower.
   - c. The period of the entire structure is not greater than 1.1 times the period of the tower considered as a separate structure fixed at the base.

104.8.3 ***Dynamic Procedure.*** The dynamic lateral force procedures of Section 106 shall be used for all other structures, including the following:

1. Structures 240 or more feet in height, except as permitted by Section 104.8.2, Item 1.
2. Structures having a stiffness, weight, or geometric vertical irregularity of Type A, B, or C as defined in Table 104-4 or structures having irregular features not described in either Table 104-4 or 104-5, except as permitted by Section 105.3.1.

3. Structures over five stories or 65 feet in height in Seismic Zones 3 and 4 not having the same structural system throughout their height, except as permitted by Section 105.3.1.
4. Structures, regular or irregular, located on Soil Profile Type S4 that have a period greater than 0.7 seconds. The analysis shall include the effects of the soils at the site and shall conform to Section 106.2.4.

104.9 System Limitations

Limits are placed on the use of some structural systems, in accordance with the Requirements of this section.

104.9.1 ***Discontinuity.*** Structures, other than wood frame structures, with a discontinuity in capacity corresponding to vertical irregularity Type E as defined in Table 104-4, shall not be permitted over two stories or 30 feet in height where the "weak" story has a calculated strength of less than 65 percent of the story above.

> *Exception*: Where the "weak" story is capable of resisting a total lateral seismic force of $3(R_w/8)$ times the design force prescribed in Section 105.

104.9.2 ***Undefined Structural Systems.*** The dynamic characteristics of undefined structural systems shall be established based on tests and other technical data. The lateral force resistance and energy absorption capacity shall be substantiated as equivalent to the systems listed in Table 104-6 if the equivalent R_w values listed in Table 104-6 are used.

104.9.3 ***Irregular Features.*** All structures having irregular features described in Table 104-4 or 104-5 shall be designed to meet the additional requirements of those sections referenced in the tables.

104.10 Alternative Procedures

Other methods of design for seismic forces or other systems for providing resistance to seismic forces may be used.

104.10.1 ***General.*** Alternative lateral force design procedures using rational analyses based on well-established principles of mechanics may be used in lieu of those prescribed in these Requirements.

104.10.2 ***Seismic Isolation.*** Seismic isolation, energy dissipation, and damping systems may be used, where special details are used, to provide results at least equivalent to those obtained using these Requirements for conventional structural systems. For design procedures of seismic isolation systems refer to Section 150, General Requirements for the Design and Construction of Seismic-Isolated Structures.

105 UBC §1628 Minimum Design Lateral Forces and Related Effects

105.1 General

105.1.1 Seismic forces may come from any horizontal direction.

105.1.2 The design seismic forces may be assumed to act noncurrently in the direction of each principal axis of the structure, except as required by Section 108.1.3.

105.1.3 Seismic dead load, W, is the total dead load and applicable portions of other loads listed below.

1. In storage and warehouse occupancies, a minimum of 25 percent of the floor live load shall be applicable.
2. Where partition load is used in the floor design, the applicable portion of the load shall be not less than 10 psf.
3. Where the design snow load is greater than 30 psf, the snow load shall be included. Where considerations of siting, configuration and load duration warrant, the snow load may be reduced up to 75 percent when approved by the Building Official.
4. Total weight of permanent equipment shall be included.

105.2 Static Force Procedure

105.2.1 ***Design Base Shear.*** The total design base shear in a given direction shall be determined from the following equation:

$$V = \frac{ZIC}{R_w} W \qquad \textbf{Eqn. 105-1}$$

where:
Z = Seismic zone factor given in Table 104-1
I = Importance factor given in Table 104-3
R_w = Numerical coefficient given in Table 104-6
W = The total seismic dead load defined in Section 105.1.3
C = Numerical coefficient determined from the following equation:

$$C = \frac{1.25S}{T^{2/3}} \qquad \textbf{Eqn. 105-2}$$

where:
S = Site coefficient for soil characteristics given in Table 104-2
T = Fundamental period of vibration, in seconds, of the structure for the direction under consideration determined in accordance with Section 105.2.2

The value of C need not exceed 2.75 and this value may be used for any structure without regard to soil type or structure period. Except for those requirements where code prescribed forces are scaled up by $3(R_w/8)$, the minimum value of the ratio C/R_w shall be 0.075.

105.2.2 ***Structure Period.*** The value of T shall be determined by one of the following methods:

105.2.2.1 ***Method A.*** For all buildings, the value of T may be approximated from the following equation:

$$T = C_t(h_n)^{3/4}$$

Eqn. 105-3

where: C_t = 0.035 for steel moment resisting frames

C_t = 0.030 for reinforced concrete moment resisting frames and eccentric braced steel frames

C_t = 0.020 for all other structures

Alternatively, the value of C_t for structures with concrete or masonry shear walls may be taken as $0.1/\sqrt{A_c}$. The value of A_c shall be determined from the following equation:

$$A_c = \sum A_e[0.2 + (D_e/h_n)^2]$$

Eqn. 105-4

The value of D_e/h_n for use in Equation 105-4 shall not exceed 0.9.

105.2.2.2 ***Method B.*** The fundamental period T may be calculated using the structural properties and deformational characteristics in a properly substantiated analysis. This Requirement may be satisfied by using the following equation:

$$T = 2\pi\sqrt{\left(\sum_{i=1}^{n} w_i\delta_i^2\right)\div\left(g\sum_{i=1}^{n} f_i\delta_i\right)}$$

Eqn. 105-5

The values of f_i represent any lateral force distributed approximately in accordance with the principles of Equations 105-6 through 105-8, or any other rational distribution. The elastic deflections, δ_i, shall be calculated using the applied lateral forces, f_i. T from Method B shall not be over 30 percent greater than the value of T obtained from Method A in Seismic Zone 4 and 40 percent in Seismic Zones 1, 2, and 3.

105.3 Combinations of Structural Systems

Where combinations of structural systems are incorporated into the same structure, the following Requirements shall be satisfied.

105.3.1 ***Vertical Combinations.*** The value of R_w used in the design of any story shall be less than or equal to the value of R_w used in the given direction for the story above.

Exception: This Requirement need not apply to a story where the dead weight above that story is less than 10 percent of the total dead weight of the structure.

Structures may be designed using the procedures in Section 105.3 under the following conditions:

1. The entire structure is designed using the lowest R_w value of the lateral force resisting systems used.

   or:

2. The following two-stage analysis procedure may be used for structures conforming to Section 104.8.2, Item 4:
   a. The tower portion shall be designed as a separate structure, supported laterally by a platform, using the appropriate value of R_w.
   b. The platform shall be designed as a separate structure using the appropriate value of R_w. The reactions of the tower portion shall be increased by the ratio of the R_w values of the two portions. These factored reactions shall be applied at the top of the platform in addition to the forces determined for the platform itself.

105.3.2 *Combinations Along Different Axes*

1. In Seismic Zones 3 and 4 where a structure has a bearing wall system in only one direction, the value of R_w used for design in the orthogonal direction shall not be greater than that used for the bearing wall system.
2. Any combination of bearing wall systems, dual systems, or moment resisting frame systems may be used to resist design seismic forces in structures less than 160 feet in height. Only combinations of dual systems and special moment resisting frames can be used to resist design seismic forces in structures exceeding 160 feet in height in Seismic Zones 3 and 4.

105.3.3 ***Combinations Along the Same Axes.*** For other than dual systems, where a combination of different structural systems is utilized to resist lateral forces in the same direction, the value of R_w used in that direction shall not be greater than the least value of any of the systems utilized in that same direction.

105.4 Vertical Distribution of Force

The total force shall be distributed over the height of the structure in conformance with Equations 105-6 through 105-8 in the absence of a more rigorous procedure.

$$V = F_t + \sum_{i=1}^{n} F_i$$ **Eqn. 105-6**

The concentrated force, F_t, at the top, which is in addition to F_n, shall be determined from the equation:

$$F_t = 0.07TV$$ **Eqn. 105-7**

The value of T used for the purpose of calculating F_t may be the period that corresponds with the design base shear, as computed using Equation 105-1. F_t need not exceed 0.25V and may be considered as zero (0) where T is 0.7 seconds or less. The remaining portion of the base shear shall be distributed over the height of the structure, including level n, according to the following equation:

$$F_x = \frac{(V - F_t) w_x h_x}{\sum_{i=1}^{n} w_i h_i} \qquad \textbf{Eqn. 105-8}$$

At each level designated as x, the force F_x shall be applied over the area of the building in accordance with the mass distribution at that level. Stresses in each structural element shall be calculated as the effect of forces F_x and F_t applied at the appropriate levels above the base.

105.5 Horizontal Distribution of Shear

Shears shall be distributed as follows:

105.5.1 The design story shear, V_x, in any story is the sum of the forces F_x and F_t above that story. V_x shall be distributed to the various elements of the vertical lateral force resisting system in proportion to their rigidities, considering the rigidity of the diaphragm (see Section 108.2.4 for rigid elements that are not intended to be part of the lateral force resisting system).

105.5.2 Where the diaphragms are not flexible, the mass at each level shall be assumed to be displaced from the calculated center of mass in each direction a distance equal to 5 percent of the building dimension at that level perpendicular to the direction of the force under consideration. The effect of this displacement on the story shear distribution shall be considered.

105.5.3 Diaphragms shall be considered flexible for the purposes of distribution of story shear and torsional moment when the maximum lateral deformation of the diaphragm is more than two times the average story drift of the associated story. This may be determined by comparing the computed midpoint in-plane deflection of the diaphragm itself under lateral load with the story drift of adjoining vertical resisting elements under equivalent tributary lateral load.

105.6 Horizontal Torsional Moments

105.6.1 Provision shall be made for the increased shears resulting from horizontal torsion where diaphragms are not flexible. The most severe load combination for each element shall be considered in design.

105.6.2 The torsional design moment at a given story shall be the moment resulting from eccentricities between applied design lateral forces at levels above that story and the vertical resisting elements in that story plus an accidental torsional moment.

105.6.3 The accidental torsional moment shall be determined assuming the mass is displaced as per Section 105.5.2.

105.6.4 Where torsional irregularity exists (plan irregularity type A, as defined in Table 104-5), the effects shall be accounted for by increasing the accidental torsion at each level by an amplification factor, A_x, determined from the following equation:

$$A_x = \left[\frac{\delta_{max}}{1.2\delta_{avg}}\right]^2 \qquad \textbf{Eqn. 105-9}$$

where: δ_{max} = Maximum displacement at Level x

δ_{avg} = Average of the displacements at extreme points of the building at Level x

A_x need not exceed 3.0

105.7 Overturning

Every structure shall be designed to resist the overturning effects caused by earthquake forces specified in Section 105.4. At any level, the overturning moments to be resisted shall be determined using those seismic forces (F_t and F_x) that act on levels above the level under consideration. At any level, the incremental changes of the design overturning moment shall be distributed to the various resisting elements in the manner prescribed in Section 105.5, above. Overturning effects on every element shall be carried down to the foundation (see Section 108.1 for combining gravity and seismic forces).

105.7.1 Redistribution of overturning effects may be made to other vertical members if framing members of sufficient strength and stiffness to transmit the required loads are provided.

105.7.2 In Seismic Zones 2, 3, and 4, where a lateral force resisting element is discontinuous, such as for vertical irregularity type D in Table 104-4 or plan irregularity type D in Table 104-5, columns supporting such discontinuous elements shall have the strength to resist the axial forces resulting from the combination of expected gravity loads and amplified design seismic loads in addition to all other load combinations. Expected maximum and minimum gravity loads in combination with amplified design seismic shall be determined from the following equations:

$$U = 1.0DL + 0.8LL + 3(R_w/8)E \qquad \textbf{Eqn. 105-10}$$

$$U = 0.85DL \pm 3(R_w/8)E \qquad \textbf{Eqn. 105-11}$$

105.7.2.1 The axial forces in such columns need not exceed the capacity of other elements of the structure to transfer these loads to the column.

105.7.2.2 Such columns shall be capable of carrying the above described axial forces without exceeding the axial strength of the column. For designs using working stress methods, this strength may be determined using an allowable stress increase of 1.7. The additional one third allowable stress increase traditionally allowed for combinations of gravity and seismic loads is not applicable.

105.7.2.3 Such columns shall meet the following detailing Requirements or member limitations:

1. Chapter 4 for concrete in Seismic Zones 2, 3, and 4.
2. Chapter 7 for steel in Seismic Zones 2, 3, and 4.
3. See Section 108.2.10 for overturning moments to be resisted at the foundation-soil interface. For regular buildings, the force F_t may be omitted when determining the overturning moment to be resisted at the foundation-soil interface.

105.8 Story Drift Limitation

Story drift is the displacement of one level relative to the level above or below caused by the design lateral forces. Calculated drift shall include translational and torsional deflections.

105.8.1 Calculated story drift shall not exceed $0.04/R_w$ or 0.005 times the story height for structures having a fundamental period of less than 0.7 seconds. For structures having a fundamental period of 0.7 seconds or greater, the calculated story drift shall not exceed $0.03/R_w$ or 0.004 times the story height. The period used in this determination shall be the same as that used for determining the base shear.

105.8.2 These drift limits may be exceeded where it is demonstrated that greater drift can be tolerated by both structural elements and nonstructural elements that could affect life safety.

105.8.3 The design lateral forces used to determine the calculated drift may be derived from a value of C resulting from a period determined from Section 105.2.2.2 Method B, neglecting the 30 and 40 percent limitations of Section 105.2.2.2 and the lower bound limit of 0.075 for C/R_w.

105.9 P-delta Effects

The resulting member forces and moments and the story drifts induced by P-delta effects shall be considered in the evaluation of overall structural frame stability. P-delta need not be considered when the ratio of secondary moment to primary moment does not exceed 0.10; the ratio may be evaluated for any story as the product of the total dead, floor live load, and the snow load (as required in UBC (§1603.6) above the story times the seismic drift in that story, divided by the product of the seismic shear in that story, times the height of that story. In Seismic Zones 3 and 4, P-delta need not be considered where the story drift ratio does not exceed $0.02/R_w$.

105.10 Vertical Component of Seismic Forces

The following Requirements apply in Seismic Zones 3 and 4 only.

105.10.1 Horizontal cantilever components shall be designed for a net upward force of $0.5\ Z\ W_p$.

105.10.2 In addition to all other applicable load combinations, horizontal prestressed components shall be designed using not more than 50 percent of the dead load for the gravity load, alone or in combination with the lateral force effects.

106 UBC §1629 Dynamic Lateral Force Procedure

106.1 General

Dynamic analyses procedures, where used, shall conform to the criteria established in this section. The analysis shall be based on ground motions defined using the procedures given in Section 106.2 and shall be performed using established principles of mechanics. Structures designed in accordance with this section shall comply with all other applicable provisions of these Requirements.

106.2 Ground Motion

The ground motion representation shall, as a minimum, have a 10 percent probability of exceedance in 50 years and may be one of the following:

106.2.1 ***Normalized Response Spectra.*** The normalized response spectrum shape is given in Figure 104-2 for the site soil type. The spectrum shape is scaled by the Z factor for the seismic zone.

106.2.2 ***Site-Specific Response Spectra.*** A site-specific response spectrum shall be based on the geologic, tectonic, seismologic, and soil characteristics associated with the specific site. The spectra shall be developed for a damping ratio of 0.05 unless a different value is shown to be consistent with the anticipated structural behavior at the intensity of shaking established for the site.

106.2.3 ***Time Histories.*** Ground motion time histories developed for the specific site shall be representative of actual earthquake motions. Response spectra from time histories, either individually or in combination, shall approximate the site-specific design spectra conforming to Section 106.2.2, above.

106.2.4 ***Structures on Soil Profile Type S4.*** The following Requirements shall apply when required by Section 104.8.3:

1. Ground motion representation shall be developed in accordance with Sections 106.2.2 and 106.2.3, above, and shall equal or exceed the motion having a 10 percent probability of exceedance in 50 years.
2. Possible amplification of building response caused by soil-structure interaction and lengthening of building period caused by inelastic behavior shall be considered.
3. Base shear determined by these procedures may be reduced to a design base shear, V, by dividing by a factor not greater than the appropriate R_w factor for the structure, but shall not be less than required by Section 106.4.1.3.1.

106.2.5 ***Vertical Component.*** The vertical component of ground motion may be defined by scaling the corresponding adjusted horizontal accelerations by a factor of two thirds. Alternative factors may be used when substantiated by site-specific data.

106.3 Mathematical Model

A mathematical model of the physical structure shall represent the spatial distribution of the mass and stiffness of the structure to an extent adequate for the calculation of the significant features of its dynamic response. A three-dimensional model shall be used for the dynamic analysis of structures with highly irregular plan configurations, such as those having a plan irregularity defined in Table 104-5 and having a rigid or semi-rigid diaphragm.

106.4 Dynamic Analysis Methods

Dynamic analysis procedures may be either an elastic response spectrum analysis method, or an elastic or inelastic time-history analysis method.

106.4.1 ***Response Spectrum Analysis.*** A response spectrum analysis is an elastic dynamic analysis of a structure that uses the peak dynamic response of all modes that have a significant contribution to total structural response. Peak modal responses are calculated using the ordinates—which correspond to the modal periods—of the appropriate response spectrum curve. Maximum modal contributions are combined in a statistical manner to obtain an approximate total structural response.

106.4.1.1 ***Number of Modes.*** At least 90 percent of the participating mass of the structure shall be included in the calculation of response for each principal horizontal direction. The minimum number of modes to be used is that sufficient to provide the 90 percent mass participation.

106.4.1.2 ***Combining Modes.*** The peak member forces, displacements, story forces, story shears, and base reactions for each mode shall be combined using established procedures in order to estimate resultant maximum values of these response parameters. When three-dimensional models are used for analysis, modal interaction effects shall be considered when combining modal maxima.

106.4.1.3 ***Scaling of Results.*** Where the base shear for a given direction, determined using these procedures, is different than the base shear by the procedures in Section 105, the adjustment procedures given below shall be followed. All corresponding response parameters, including member forces and moments, shall be adjusted proportionately. Deflection adjustments shall conform to Section 105.8.3.

106.4.1.3.1 When the base shear is less than that determined from Section 105, it shall be increased to the following percentage of the value from Section 105:

1. 100 percent for irregular structures.
2. 90 percent for regular structures except that the base shear shall not be less than 80 percent of that determined from Section 105.2 using *T* from Section 105.2.2.1.

106.4.1.3.2 When the base shear is greater than that determined from Section 105, the value need not exceed that required by Section 106.4.1.3.1 above, except for structures required to conform to Section 106.2.4.

106.4.1.4 ***Directional Effects.*** Directional effects for horizontal ground motion shall conform to the Requirements of Sections 105.1.1 and 105.1.2. The effects of vertical ground motions on horizontal cantilevers and prestressed elements shall be considered in accordance with Section 105.10. Alternately, vertical seismic response may be determined by dynamic response methods; in no case shall the response used for design be less than that obtained by the static method.

106.4.1.5 ***Torsion.*** The analysis shall account for torsional effects, including accidental torsional effects as prescribed in Section 105.6. Where three-dimensional models are used for analysis, effects of accidental torsion shall be accounted for by appropriate adjustments in the model such as adjustment of mass locations, or by equivalent static procedures such as provided in Section 105.6.

106.4.1.6 ***Dual Systems.*** Where the lateral forces are resisted by a dual system as defined in Section 104.6.4, the system shall satisfy the following Requirements:

1. The combined system shall be capable of resisting the base shear determined in accordance with this section.
2. The SMRF shall conform to Section 104.6.4.2 and may be analyzed using either the procedures of Section 105.4 or those of this section.

106.5 Time-History Analysis

A time-history analysis is an elastic or inelastic dynamic analysis in which a mathematical model of the structure is subjected to a specified ground motion time history. The structure's time-dependent dynamic response to these motions is obtained by numerical integration of its equations of motion. Time-history analyses shall meet the Requirements of Section 104.10.

107 UBC §1630 Lateral Force on Elements of Structures, Nonstructural Components, and Equipment Supported by Structures

107.1 UBC §1630.1 General

Elements of structures and their attachments, permanent nonstructural components and their attachments, and the attachments for permanent equipment supported by a structure shall be designed to resist the total design seismic forces prescribed in Section 107.2. Attachments for floor and roof mounted equipment weighing less than 400 pounds (181 kg) and furniture need not be detailed on construction drawings or shown in design calculations.

107.1.1 ***Attachments.*** Attachments shall include anchorages and required bracing. Friction resulting from gravity loads shall not be considered to provide resistance to seismic forces.

107.1.2 ***Equipment Design.*** When the structural failure of the lateral force resisting systems of nonrigid equipment would cause a life hazard, such systems, as well as the equipment itself, shall be designed to resist the total design seismic forces prescribed in Section 107.

107.1.3 ***Equipment Support Design.*** When allowable design stresses and other acceptance criteria are not contained in or referenced by these Requirements, such criteria shall be obtained from approved national standards

107.2 UBC §1630.2 Total Design Seismic Force

Each element or component specified in Section 107.1 shall be designed to resist the total lateral design seismic force, F_p, determined by the following equation:

$$F_p = Z I_p C_p C_f C_g W_p \qquad \textbf{Eqn. 107-1}$$

107.2.1 The values of Z and I_p shall be the values of Z and I used for the structure.

Exceptions:

1. For essential or hazardous facilities, the value of I_p shall be 1.5. For anchorage of machinery and equipment required for life safety systems, the value of I_p shall be 1.5.
2. For the design of tanks and vessels containing sufficient quantities of highly toxic or explosive substances to be hazardous to the safety of the general public if released, the value of I_p shall be 1.5.
3. For items that block a means of egress or exit way if damaged, I_p shall be 1.5.
4. The value of I_p for panel connectors shall be 1.0 for the entire connection (see Section 108.2.4.2).

107.2.2 The coefficient, C_p, is used for elements and components and for rigid and rigidly supported equipment. Rigid or rigidly supported equipment is defined as having a fundamental period less than or equal to 0.06 seconds. Flexible or flexibly supported equipment is defined as a system having a fundamental period greater than 0.06 seconds. The flexibility of the equipment, equipment support, and equipment anchorages and attachments shall be included in the determination of whether the equipment is considered flexible, flexibly supported, rigid, or rigidly supported. The value of C_p shall not be less than that listed in Table 104-7. Piping, ducting, and conduit systems constructed of ductile materials and that have ductile connections may use the values of C_p from Table 104-7.

107.2.3 The value of C_f, the flexibility coefficient, for flexible or flexibly supported equipment supported by a structure or located above grade, shall be determined considering the dynamic properties of both the equipment and the supporting structure. In the absence of an analysis or other substantiating data, the value of C_f for flexible or flexibly supported equipment shall be taken as 2.0. The value of C_f for rigid equipment and all elements and components shall be not less than 1.0.

107.2.4 The value of C_g, the at-grade coefficient, shall be not less than 1.0 for elements, components, and rigid or flexible equipment supported by a structure above grade. The value of C_g may be 0.67 for elements, components, and rigid or flexible equipment laterally self-supported at or below grade. However, the design lateral forces for an element, component, or equipment shall be not less than the forces that would be determined by considering the item as an independent structure and using the Requirements of Section 109.

107.2.5 The design lateral forces determined using Equation 107-1 shall be distributed proportional to the mass distribution of the element, component, or equipment.

107.2.6 Forces determined using Equation 107-1 shall be used to design elements and components, their connections and anchorage to the structure, and members and connections that transfer these forces to the lateral force resisting system.

107.2.7 For forces applicable to connectors for exterior panels and in diaphragms, refer to Sections 108.2.4.2 and 108.2.9, respectively.

107.2.8 Forces shall be applied in the horizontal directions that result in the most critical loadings for design.

107.3 UBC §1630.3 Specifying Lateral Forces

Design specifications for equipment shall either specify the design lateral forces prescribed herein or reference these Requirements.

107.4 UBC §1630.4 Relative Motion of Equipment Attachments

For equipment in facilities assigned to Occupancy Categories I and II structures, as set forth in Table 104-3, and for life safety systems, the lateral force design shall consider the effects of relative motion of the points of attachment to the structure (drift).

107.5 UBC §1630.5 Alternative Designs

Where an approved national standard or approved physical test data provide a basis for the earthquake resistant design of a particular type of equipment or nonstructural component, such standard or data may be accepted as a basis for design of elements, nonstructural components, and equipment, with the following limitations:

107.5.1 These Requirements shall provide minimum values for design of the attachments and supports that transfer the forces to the seismic resisting system.

107.5.2 The force, F_p, and the overturning moment used in the design of the nonstructural component or equipment attachments and supports shall be not less than 80 percent of the values that would be obtained using these Requirements.

108 UBC §1631 Detailed Systems Design Requirements

108.1 General

All structural framing systems shall comply with the Requirements of Section 104. Only the elements of the designated seismic force resisting system shall be used to resist design forces. The individual components shall be designed to resist the prescribed design seismic forces acting on them. The components shall also comply with the specific Requirements for the material contained in Chapters 4 through 8. In addition, such framing systems and components shall comply with the detailed system design Requirements contained in this section.

108.1.1 ***Combined Vertical and Horizontal Forces.*** All building components shall be designed to resist the effects of the seismic forces prescribed herein and the effects of gravity loadings from dead, floor live, and snow loads.

108.1.2 ***Uplift Effects.*** Consideration shall be given to design for uplift effects caused by seismic loads. For materials in which working stress procedures are used for design, dead loads used to reduce uplift shall be multiplied by 0.85.

108.1.3 ***Orthogonal Effects***

108.1.3.1 In Seismic Zones 2, 3, and 4 provision shall be made for the effects of earthquake motions acting in directions other than parallel to the direction of resistance under consideration in each of the following circumstances.

1. The structure has plan irregularity Type E as given in Table 104-5.
2. The structure has plan irregularity Type A as given in Table 104-5 for both major axis directions.
3. A column of a structure forms part of two or more intersecting lateral force resisting systems.

   Exception: If the axial load in the column due to seismic forces acting in either direction is less than 20 percent of the allowable column axial strength.

108.1.3.2 The requirement that orthogonal effects be considered may be satisfied by designing such elements for 100 percent of the prescribed seismic forces in one direction plus 30 percent of the prescribed forces in the perpendicular direction. The result that requires the greater component strength shall be used for design. Alternatively, the effects of the two orthogonal directions may be combined on a square-root-sum-of-the-squares (SRSS) basis. When the SRSS method of combining directional effects is used, each term computed shall be assigned the sign that will result in the largest force on the member.

108.2 Structural Framing Systems

108.2.1 *General.* Four general types of building framing systems defined in Section 104.6 are recognized in these Requirements and shown in Table 104-6. Each type is subdivided by the types of vertical elements used to resist lateral seismic forces. Special framing Requirements are given in this section and in Chapters 4 through 8.

108.2.2 *Detailing Requirements for Combinations of Systems.* For components common to different structural systems, the more restrictive detailing Requirements shall be used.

108.2.3 *Connections.* Connections that resist prescribed seismic forces shall be designed and detailed on the drawings.

108.2.4 *Deformation Compatibility.* All structural framing elements and their connections, not required by design to be part of the lateral force resisting system, shall be designed and/or detailed to be adequate to maintain support of design dead plus live loads when subjected to the expected deformations caused by seismic forces. P-delta effects on such elements shall be accounted for. Expected deformations shall be estimated as the greater of $3(R_w/8)$ times the computed elastic deformations at design seismic forces, or the deformation induced by a story drift of 0.0025 times story height. When computing expected deformations, the stiffening effect of those elements not part of the lateral force resisting system shall be neglected.

The forces induced by the expected deformation may be considered as ultimate or factored forces. When computing the forces induced by expected deformations, the restraining effect of adjoining rigid structural and nonstructural elements shall be considered and a rational value of member and restraint stiffness shall be used. Inelastic deformations of members and connections may be considered in the evaluation, provided capacities are consistent with member and connection design and detailing.

For concrete and masonry elements, flexural and shear stiffness properties shall not exceed one-half of the gross section properties unless a rational cracked-section analysis is performed. Additional deformations, which may result from foundation flexibility and diaphragm deflections, shall be considered. For concrete elements not part of the lateral force resisting system, see Section 402.13.

108.2.4.1 *Adjoining Rigid Elements.* In Seismic Zones 3 and 4, moment resisting frames and shear walls may be enclosed by or adjoined by more rigid elements provided it can be shown that the participation or failure of the more rigid elements will not impair the vertical and lateral load-resisting ability of the gravity load and lateral force resisting systems. The effects of adjoining rigid elements shall be considered when assessing whether a structure shall be designated regular or irregular in Section 104.5.

108.2.4.2 ***Exterior Elements.*** Exterior nonbearing, nonshear wall panels or elements that are attached to or enclose the exterior of the structure shall be designed to resist the forces per Equation 107-1 and shall accommodate movements of the structure resulting from lateral forces or temperature changes. Such elements shall be supported by structural members or by mechanical connections and fasteners joining them to structural members in accordance with the following Requirements:

1. Connections and panel joints shall allow for a relative movement between stories of not less than two times story drift caused by wind, $3(R_w/8)$ times the calculated story drift caused by design seismic forces, or 1/2-inch, whichever is greater.
2. Connections to permit movement in the plane of the panel for story drift shall be sliding connections using slotted or oversize holes, connections that permit movement by bending of steel, or other connections providing equivalent sliding and ductility capacity.
3. Bodies of connections shall have sufficient ductility and rotation capacity to preclude fracture of the anchoring elements or brittle failures at or near welds.
4. The body of the connection shall be designed for 1-1/3 times the force determined by Equation 107-1.
5. All fasteners in the connection system, such as bolts, inserts, welds, dowels, etc., shall be designed for four times the forces determined by Equation 107-1.
6. Fasteners embedded in concrete shall be attached to, or hooked around, reinforcing steel or otherwise terminated so as to effectively transfer forces to the reinforcing steel.

108.2.5 *Ties and Continuity*

1. All parts of a structure shall be interconnected. These connections shall be capable of transmitting the prescribed seismic force induced by the parts being connected to the seismic resisting system. As a minimum, any smaller portions of a building shall be tied to the remainder of the building with elements having the strength to transfer at least $Z/3$ times the weight of the smaller portion.
2. A positive connection for resisting a horizontal force acting parallel to the member shall be provided for each beam, girder, or truss. This force shall be not less than $Z/5$ times the sum of the dead and live load tributary to the member.

108.2.6 ***Collector Elements.*** Collector elements shall be provided that are capable of transferring the seismic forces originating in other portions of the building to the element providing the resistance to those forces.

108.2.7 ***Concrete Frames.*** Concrete frames required by design to be part of the lateral force resisting system shall conform to the following:

1. In Seismic Zones 3 and 4 they shall be special moment resisting frames.
2. In Seismic Zone 2 they shall, as a minimum, be intermediate moment resisting frames.

108.2.8 ***UBC §1631.2.8 Anchorage of Concrete or Masonry Walls.*** Concrete or masonry walls shall be anchored to all floors and roofs that provide out-of-plane lateral support of the wall. The anchorage shall provide a positive direct connection between the wall and floor or roof construction capable of resisting the horizontal forces specified in this section, or a minimum of 200 pounds per lineal foot of wall, whichever is greater. Walls shall be designed to resist bending between anchors where the anchor spacing exceeds 4 feet. In masonry walls of hollow units or cavity walls, anchors shall be embedded in a reinforced grouted structural element in the wall. Requirements for developing anchorage forces in diaphragms are given in Section 108.2.9, below. Diaphragm deformations shall be considered in the design of the supported walls.

108.2.8.1 ***Out-of-Plane Wall Anchorage to Flexible Diaphragms.*** This section shall apply in Seismic Zones 3 and 4 where flexible diaphragms, as defined in Section 105.5, provide lateral support for walls.

1. Elements of the wall anchorage system shall be designed for the forces specified in Section 107, using a C_p value of 1.2. In Seismic Zone 4, the value of F_p used for the design of elements of the wall anchorage system shall not be less than 300 pounds per lineal foot (4.38 KN/m) of wall.
2. When elements of the wall anchorage system are not loaded concentrically or are not perpendicular to the wall, the system shall be designed to resist all components of forces induced by the eccentricity.
3. When pilasters are present in the wall, the anchorage force at the pilasters shall be calculated considering that the wall panels between the pilasters are supported on four sides. However, the minimum anchorage force at a floor or roof shall be that specified in Section 108.2.8.1, Item 1.
4. Strength design, using a load factor of 1.7 for E, shall be used for design of embedment in concrete.
5. The working stress design forces for steel elements of the wall anchorage system shall be increased by a factor of 1.7 over the forces otherwise required by this section.
6. Wood elements of the wall anchorage system shall have a minimum actual net thickness of 2.5 inches (62 mm).

In addition, in Seismic Zones 3 and 4, diaphragm to wall anchorage using embedded straps shall have the straps attached to or hooked around the reinforcing steel or otherwise terminated so as to effectively transfer forces to the reinforcing steel.

108.2.9 *Diaphragms*

108.2.9.1 ***Deflections.*** The deflection in the plane of the diaphragm shall not exceed the permissible deflection of the attached elements. Permissible deflection shall be that deflection that will permit the attached element to maintain its structural integrity under the individual loading and continue to support the prescribed loads.

108.2.9.2 ***Diaphragm Forces.*** Floor and roof diaphragms shall be designed to resist the forces determined in accordance with the following equation:

$$F_{px} = \left(\frac{F_t + \sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} \right) w_{px} \qquad \textbf{Eqn. 108-1}$$

1. The force F_{px} determined from Equation 108-1 need not exceed 0.75 $Z\,I\,w_{px}$, but shall not be less than 0.35 $Z\,I\,w_{px}$.
2. When the diaphragm is required to transfer lateral forces from the vertical resisting elements above the diaphragm to other vertical resisting elements below the diaphragm due to offset in the placement of the elements or to changes in stiffness in the vertical elements, these forces shall be added to those determined from Equation 108-1.

108.2.9.3 ***Design Forces for Flexible Diaphragms.*** Design forces for flexible diaphragms, providing lateral supports for walls or frames of masonry or concrete, shall be calculated using an R_w not to exceed 6.

108.2.9.4 ***Diaphragm Ties.*** Diaphragms supporting concrete or masonry walls shall have continuous ties or struts between diaphragm chords to distribute the anchorage forces specified in Section 108.2.8. The spacing of the continuous ties or struts shall be the greater of the girder spacing or 24 feet. Added chords of subdiaphragms may be used to form subdiaphragms to transmit the anchorage forces to the continuous ties. The maximum length to width ratio of the subdiaphragm shall be 2 1/2 to 1.

108.2.9.5 ***Wood Diaphragms Used to Support Concrete or Masonry Walls.*** Where wood diaphragms are used to laterally support concrete or masonry walls, the anchorage shall conform to Section 108.2.8.1, above. In Seismic Zones 2, 3, and 4 the following Requirements apply:

1. Anchorage shall not be accomplished by use of toe nails or nails subject to withdrawal; nor shall wood ledgers or framing be used in cross-grain bending or cross-grain tension.
2. The continuous ties required by Section 108.2.9.4, above, shall be in addition to the diaphragm sheathing.

108.2.9.6 ***Plan Irregularity.*** For structures in Seismic Zones 3 and 4 having plan irregularity of Types A, B, C, or D in Table 104-5, connections of diaphragms to the vertical elements and to collectors, and connections of collectors to the vertical elements shall be designed without considering the one-third increase usually permitted in allowable stresses for elements resisting earthquake forces.

108.2.9.7 ***Re-Entrant Corners.*** For structures in Seismic Zones 3 and 4 having plan irregularity of Type B in Table 104-5, diaphragm chords and collectors shall be designed considering independent movement of the projecting wings of the structure. Each of these diaphragm elements shall be designed for the more severe of the following two assumptions:

1. Motion of the projecting wings in the same direction.
2. Motion of projecting wings in opposing directions.

> *Exception*: This Requirement may be deemed satisfied if the procedures of Section 106, in conjunction with a three-dimensional model, have been used to determine the lateral seismic forces for design.

108.2.10 ***Framing Below the Base***

1. **General.** The strength and stiffness of framing between the base and the foundation shall not be less than that of the superstructure. The special detailing Requirements of Chapters 3 and 4, as appropriate, shall apply to columns supporting discontinuous lateral force resisting elements and to SMRF, IMRF, and EBF system elements below the base that are required to transmit the forces resulting from lateral loads to the foundation.
2. **Foundations.** The foundation shall be capable of transmitting the design base shear and the overturning forces defined in Section 105 from the structure into the supporting soil, but the short-term dynamic nature of the loads may be taken into account in establishing the soil properties. Chapter 3 defines additional Requirements for specific types of foundation construction.

108.2.11 ***Building Separations.*** All structures shall be separated from adjoining structures. Separations shall allow for $3(R_w/8)$ times the displacement due to design seismic forces. When a structure adjoins a property line not common to a public way, that structure shall also be set back from the property line by at least $3(R_w/8)$ times the displacement of the structure.

> *Exception*: Smaller separations may be permitted when justified by rational analyses based on maximum expected ground motions. As a minimum, building separations shall not be less than $(R_w/8) \geq 1$ times the sum of displacements due to code-specified seismic forces.

108.2.12 ***Boundary Members.*** Specially detailed boundary members shall be considered for shear walls and shear wall elements whenever their design is governed by flexure.

109 UBC §1632 Nonbuilding Structures

109.1 General

109.1.1 ***Scope.*** Nonbuilding structures include all self-supporting structures, other than buildings, that carry gravity loads and resist the effects of earthquakes. Nonbuilding structures shall be designed to resist the minimum lateral forces specified in these Requirements. Design shall conform to the applicable provisions of other sections of these Requirements, modified by the provisions contained in this section.

109.1.2 ***Criteria.*** The minimum design lateral forces prescribed in these Requirements are at a service level (rather than yield or ultimate level). The design of nonbuilding structures shall provide sufficient strength and ductility, consistent with these Requirements for buildings, to resist the effects of seismic ground motions as represented by the design forces set forth in this section.

109.1.2.1 When applicable, allowable stresses and other detailed design criteria shall be obtained from other sections of these Requirements, or codes and standards referenced herein.

109.1.2.2 When applicable design stresses and other design criteria are not contained in or referenced by these Requirements, such criteria shall be obtained from approved national standards.

109.1.3 ***Weight W.*** The weight *W* for nonbuilding structures shall include all seismic dead load, as defined for buildings in Section 105.1.3. For purposes of calculating design seismic forces in nonbuilding structures, *W* shall also include all normal operating contents for items such as tanks, vessels, bins, and piping.

109.1.4 ***Period.*** The fundamental period of the structure shall be determined by rational methods such as by using Method B in Section 105.2.2.

109.1.5 ***Drift.*** The drift limitations of Section 105.8 need not apply to nonbuilding structures. Drift limitations shall be established for structural or nonstructural elements whose failure would cause life hazards. P-delta effects shall be considered for structures whose calculated drifts exceed the values in Section 105.8.

109.1.6 ***Interaction Effects.*** In Seismic Zones 3 and 4, structures that support flexible nonstructural elements whose combined weight exceeds 25 percent of the weight of the structure shall be designed considering interaction effects between the structure and the supported elements.

109.2 Lateral Force

The lateral force procedure for nonbuilding structures with structural systems similar to buildings (those with structural systems listed in Table 104-6) shall be selected in accordance with the Requirements of Section 104.

> *Exception*: Intermediate moment resisting frames (IMRF) may be used in Seismic Zones 3 and 4 for nonbuilding structures in Occupancy Categories III and IV if: 1) the structure is less than 50 feet in height, and 2) an $R_w = 4.0$ is used for design.

109.3 Rigid Structures

Rigid structures (those with period T less than 0.06 seconds), including their anchorages, shall be designed for the lateral force obtained from:

$$V = 0.5ZIW \qquad \textbf{Eqn. 109-1}$$

The force V shall be distributed according to the distribution of mass and shall be assumed to act in any horizontal direction.

109.4 Tanks With Supported Bottoms

Flat-bottom tanks or other tanks with supported bottoms, founded at or below grade, shall be designed to resist the seismic forces calculated using the procedures in Section 109.3, above, for rigid structures considering the entire weight of the tank and its contents. Alternatively, such tanks may be designed using one of the two procedures described below.

1. A response spectrum analysis, which includes consideration of the actual ground motion anticipated at the site and the inertial effects of the contained fluid.
2. A design basis prescribed for the particular type of tank by an approved national standard, provided that the seismic zones and occupancy categories shall be in conformance with the Requirements of Sections 104.2 and 104.4, respectively.

109.5 Other Nonbuilding Structures

Nonbuilding structures that are not covered by Sections 109.2 through 109.4, above, shall be designed to resist minimum seismic lateral forces of not less than those determined in accordance with the Requirements in Section 105 with the following additions and exceptions:

109.5.1 The factor R_w shall be as given in Table 109-1. The ratio C/R_w used for design shall be not less than 0.4.

109.5.2 The vertical distribution of the lateral seismic forces in structures covered by this section may be determined in one of the two following methods.

1. Using the Requirements of Section 105.4.
2. Using the procedures of Section 106.

   ***Exception*:** For irregular structures assigned to Occupancy Categories I and II that cannot be modeled as a single mass, the procedures of Section 106 shall be used.

109.5.3 Where an approved national standard provides a basis for the earthquake resistant design of a particular type of nonbuilding structure covered by this Section 109.5, such a standard may be used subject to the following limitations.

1. The seismic zones and occupancy categories shall be in conformance with the Requirements of Sections 104.2 and 104.4, respectively.
2. The values for total lateral force and total base overturning moment used in design shall not be less than 80 percent of the values that would be obtained using these Recommendations.

110 UBC §3400 Existing Structures

110.1 UBC 3401 General

Buildings in existence at the time of the adoption of this code may have their existing use or occupancy continued, if such use or occupancy was legal at the time of the adoption of this code, provided such continued use is not dangerous to life. Any change in the use or occupancy of any existing building or structure shall comply with the Requirements of 1994 UBC §§109 and 3405.

For existing buildings, see UBC §101.3, Chapter 34, and Appendix to Chapter 34. For a comprehensive code and guidelines on the treatment of existing buildings, see *Uniform Code for Building Conservation* (UCBC).

110.2 UBC §3402 Maintenance

All buildings and structures, both existing and new, and all parts thereof, shall be maintained in a safe and sanitary condition. All devices or safeguards that are required by this code shall be maintained in conformance with the code edition under which installed. The owner or the owner's designated agent shall be responsible for the maintenance of buildings and structures. To determine compliance with this subsection, the building official may cause a structure to be reinspected.

110.3 UBC §3403 Additions, Alterations, or Repairs

110.3.1 ***General.*** Buildings and structures to which additions, alterations, or repairs are made shall comply with all the requirements of this code for new facilities, except as specifically provided in this section. See UBC §310.9 for requirements requiring installation of smoke detectors in existing Group R, Division 3 Occupancies.

110.3.2 ***When Allowed.*** Additions, alterations, or repairs may be made to any building or structure without requiring the existing building or structure to comply with all the Requirements of this code, provided the addition, alteration, or repair conforms to that required for a new building or structure.

Additions or alterations shall not be made to an existing building or structure that will cause the existing building or structure to be in violation of any of these Requirements, nor shall such additions or alterations cause the existing building or structure to become unsafe. An unsafe condition shall be deemed to have been created if an addition or alteration will cause the existing building or structure:

1. To become structurally unsafe or overloaded.
2. Will not provide adequate egress in compliance with these Requirements or will obstruct existing exits.
3. Will create a fire hazard.
4. Will reduce required fire resistance or will otherwise create conditions dangerous to human life.

Any building so altered, and which involves a change in use or occupancy, shall not exceed the height, number of stories, and area permitted for new buildings. Any building, and all new additions, shall not exceed the height, number of stories and area specified for new buildings.

Additions or alterations shall not be made to an existing building or structure when such existing building or structure is not in full compliance with these Requirements, except when such addition or alteration will result in the existing building or structure being no more hazardous based on life safety, fire safety and sanitation, than before such additions or alterations are undertaken (see also UBC §307.11.2 for Group H, Division 6 Occupancies).

> ***Exception*:** Alterations of existing structural elements or additions of new structural elements not required by UBC §3401, and which are initiated for the purpose of increasing the lateral force resisting strength or stiffness of an existing structure, need not be designed for forces conforming to these Requirements provided that an engineering analysis be submitted to show the following:
>
> 1. The capacity of existing structural elements required to resist forces is not reduced.
> 2. The lateral loading to required existing structural elements is not increased beyond their capacity.
> 3. New structural elements are detailed and connected to the existing structural elements as required by these regulations.
> 4. New or relocated nonstructural elements are detailed and connected to existing or new structural elements as required by these regulations.
> 5. An unsafe condition as defined in this section is not created.

110.3.3 ***Repairs to Buildings and Structures Damaged by Natural Disaster.*** Repair criteria for structural and nonstructural elements of all buildings and structures damaged by the occurrence of a declared natural disaster shall be determined by this section. Required repair levels shall be based on the damage ratio of the estimated value of the repairs required to restore the structural members to their pre-event condition to the estimated replacement value of the building or structure as follows:

1. When the damage ratio does not exceed 0.1 (10 percent), buildings and structures, except essential service facilities included as Category I buildings and structures in Table 104-3, shall at a minimum be restored to their pre-event condition.
2. When the damage ratio is greater than 0.1 (10 percent), but less than 0.5 (50 percent), buildings and structures, except essential service facilities included as Category I buildings and structures in Table 104-3, shall at a minimum have the damaged structural members, including all critical ties and connections associated with the damaged structural members, all structural members supported by the damaged member, and all structural members supporting the damaged members repaired and strengthened to bring them into compliance with

these Requirements. This criteria shall apply to essential service facilities when the damage ratio is less than 0.3 (30 percent).

Exception: For buildings with rigid diaphragms where the above required repair and strengthening increases the rigidity of the resisting members, the entire lateral force resisting system of the building shall be investigated. When, in the opinion of the building official, an unsafe or adverse condition has been created as a result of the increase in rigidity, the condition shall be corrected.

3. When the damage ratio is greater than 0.5 (50 percent), buildings and structures, except essential service facilities included as Category I buildings and structures in Table 104-3, shall at a minimum have the entire building or structure brought into conformance with all the structural requirements herein. This criteria shall apply to essential service facilities when the damage ratio is equal to or greater than 0.3 (30 percent).

110.3.3.1 ***Light Fixtures and Suspended Ceilings.*** Under all damage ratios, when light fixtures and the suspension system of suspended ceilings are damaged, the damaged light fixtures and suspension systems shall be repaired to fully comply with these Requirements and UBC Standard 25-2. Undamaged light fixtures and suspension systems shall have the additional support and bracing provided that is required in UBC Standard 25-2.

110.3.3.2 ***Analysis of Noncomplying Structural Member.*** When the requirements of UBC §3403.3 require the removal of an otherwise undamaged structural member because that member does not comply with these Requirements, such member may remain and be analyzed, strengthened and its connections strengthened to the lateral force levels required by these Requirements.

110.3.4 ***Nonstructural.*** Alterations or repairs to an existing building or structure that are nonstructural and do not adversely affect any structural member, or any part of the building or structure having required fire resistance, may be made with the same materials of which the building or structure is constructed.

110.3.5 ***Glass Replacement.*** The installation or replacement of glass shall be as required for new installations.

110.3.6 ***Historic Buildings.*** Repairs, alterations, and additions necessary for the preservation, restoration, rehabilitation, or continued use of a building or structure may be made without conformance to all these Requirements when authorized by the building official, provided:

1. The building or structure has been designated by official action of the legally constituted authority of its jurisdiction as having special historical or architectural significance.
2. Any unsafe conditions as described in these Requirements are corrected.
3. The restored building or structure will be no more hazardous based on life safety, fire safety, and sanitation that the existing building.

110.4 UBC §3040 Moved Buildings

Buildings or structures moved into or within the jurisdiction shall comply with these Recommendations for new buildings or structures.

110.5 UBC §3405 Change In Use

No change shall be made in the character of occupancies or use of any building that would place the building in a different division of the same group of occupancy or in a different group of occupancies, unless such building is made to comply with the Requirements of this code for such division or group of occupancy.

> *Exception*: The character of the occupancy of existing buildings may be changed subject to the approval of the building official, and the building may be occupied for purposes in other groups without conforming to all the Requirements of this code for those groups, provided the new or proposed use is less hazardous, based on life and fire risk, than the existing use.

No change in the character or occupancy of a building shall be made without a certificate of occupancy, as required in UBC §109 and in these Requirements. The building official may issue a certificate of occupancy pursuant to the intent of the above exception without certifying that the building complies with all provisions of these Requirements and with UBC §109.

111 Future Objectives and Research Requirements

See Commentary.

112–149 Reserved

150 UBC §1650 General Requirements for Seismic-Isolated Structures

Every seismic-isolated structure and every portion thereof shall be designed and constructed in accordance with the Requirements of Sections 150 through 161 and the applicable Requirements of Sections 101 through 111 of these Recommendations.

1. The lateral force resisting system and the isolation system shall be designed to resist the deformations and stresses produced by the effects of seismic ground motions, as provided in Sections 150 through 161.
2. Where wind forces prescribed by these Recommendations produce greater deformations or stresses, such loads shall be used for design in lieu of the deformations and stresses resulting from earthquake forces.

151 UBC §1651 Definitions for Seismic-Isolated Structures

The definitions of Section 102 of these Recommendations and the following apply to the Requirements of Sections 150 through 161:

Design Basis Earthquake. Defined in Section 106.2.

Design Displacement. The design basis earthquake lateral displacement required for design of the isolation system, excluding additional displacement due to actual and accidental torsion.

Effective Damping. The value of equivalent viscous damping corresponding to energy dissipated during cyclic response of the isolation system.

Effective Stiffness. The value of the lateral force in the isolation system, or an element thereof, divided by the corresponding lateral displacement.

Isolation Interface. The boundary between the upper portion of the structure, which is isolated, and the lower portion of the structure, which moves rigidly with the ground.

Isolation System. The collection of structural elements, which includes all individual isolator units, all structural elements that transfer force between elements of the isolation system, and all connections to other structural elements. The isolation system also includes the wind-restraint system if such a system is used to meet the design requirements of this section.

Isolator Unit. A horizontally flexible and vertically stiff structural element of the isolation system that permits large lateral deformations under design seismic load. An isolator unit may be used either as part of, or in addition to, the weight-supporting system of the building.

Maximum Capable Earthquake. The maximum level of earthquake ground shaking that may ever be expected at the building site within the known geological framework. In Seismic Zones 3 and 4, this intensity may be taken as the level of earthquake ground motion that has a 10 percent probability of being exceeded in a 100-year time period.

Total Design Displacement. The design basis earthquake lateral displacement, including additional displacement due to actual and accidental torsion, required for design of the isolation system, or an element thereof.

Total Maximum Displacement. Lateral displacement due to the maximum capable earthquake—including additional displacement due to actual and accidental torsion—required for verification of the stability of the isolation system or elements thereof, design of building separations, and vertical load testing of isolator unit prototypes.

Wind Restraint System. The collection of structural elements that provide restraint of the seismic-isolated structure for wind loads. The wind-restraint system may be either an integral part of an isolator unit or may be a separate device.

152 UBC §1652 Symbols and Notations for Seismic-Isolated Structures

The symbols and notations of Section 103 of these Recommendations and the following apply to the Requirements of Sections 150 through 161:

B = Numerical coefficient related to the effective damping of the isolation system as set forth in Table 154-1.

b = The shortest plan dimension of the structure, in feet, measured perpendicular to d.

D = Design displacement, in inches, at the center of rigidity of the isolation system in the direction under consideration, as prescribed by Equation 154-1.

D' = Design displacement, in inches, at the center of rigidity of the isolation system in the direction under consideration, as prescribed by Equation 155-1.

D_T = Total design displacement, in inches, of an element of the isolation system, including both translational displacement at the center of rigidity, D, and the component of torsional displacement in the direction under consideration, as specified in Section 154.3.3.

D_{TM} = Total maximum displacement, in inches, of an element of the isolation system, including both translational displacement at the center of rigidity and the component of torsional displacement in the direction under consideration, as prescribed by Equation 154-4.

d = The longest plan dimension of the structure, in feet.

e = The actual eccentricity, in feet, measured in plan between the center of mass of the structure above the isolation interface and the center of rigidity of the isolation system, plus accidental eccentricity, in feet, taken as 5 percent of the maximum building dimension perpendicular to the direction of force under consideration.

F^- = Maximum negative force in an isolator unit during a single cycle of prototype testing at a displacement amplitude of Δ^-.

F^-_{max} = Maximum negative force in an isolator unit for all cycles of prototype testing at a common displacement amplitude of Δ^-.

F^-_{min} = Minimum negative force in an isolator unit for all cycles of prototype testing at a common displacement amplitude of Δ^-.

F^+ = Maximum positive force in an isolator unit during a single cycle of prototype testing at a displacement amplitude of Δ^+.

F^+_{max} = Maximum positive force in an isolator unit for all cycles of prototype testing at a common displacement amplitude of Δ^+.

F^+_{min} = Minimum positive force in an isolator unit for all cycles of prototype testing at a common displacement amplitude of Δ^+.

k_{eff} = Effective stiffness of an isolator unit, as prescribed by Equation 161-3.

k_{max} = Maximum effective stiffness of the isolation system at the design displacement in the horizontal direction under consideration.

k_{min} = Minimum effective stiffness of the isolation system at the design displacement in the horizontal direction under consideration.

M_M = Numerical coefficient related to maximum capable earthquake response as set forth in Table 152-2.

N = Numerical coefficient related to both the proximity of the building or structure to an active fault, and fault magnitude, as set forth in Table 152-3.

R_{wI} = Numerical coefficient related to the type of lateral force resisting system above the isolation system, as set forth in Table 152-1 for seismic-isolated structures.

S_I = Numerical coefficient for site-soil profile, as set forth in Table 154-2 for seismic-isolated structures.

T_I = Period of seismic-isolated structure, in seconds, in the direction under consideration, as prescribed by Equation 154-2.

V_b = The total lateral seismic design force or shear on elements of the isolation system or elements below the isolation system, as prescribed by Equation 154-5.

V_s = The total lateral seismic design force or shear on elements above the isolation system as prescribed by Equation 154-6.

W_I = The total seismic dead load defined in Section 105.1.3. For design of the isolation system, W_I is the total seismic dead load weight of the structure above the isolation interface.

y = The distance, in feet, between the center of rigidity of the isolation system and the element of interest, measured perpendicular to the direction of seismic loading under consideration.

Δ^+ = Maximum positive displacement of an isolator unit during each cycle of prototype testing.

Δ^- = Maximum negative displacement of an isolator unit during each cycle of prototype testing.

β = Effective damping of the isolation system, as prescribed by Equation 161-2.

153 UBC §1653 Criteria Selection for Seismic-Isolated Structures

153.1 Basis for Design

The procedures and limitations for the design of seismic-isolated structures shall be determined considering zoning, site characteristics, vertical acceleration, cracked section properties of concrete and masonry members, occupancy, configuration, structural system, and height in accordance with Section 104, except as noted below.

153.2 Stability of the Isolation System

The stability of the vertical load-carrying elements of the isolation system shall be verified by analysis and test, as required, for lateral seismic displacement equal to the total maximum displacement.

153.3 Occupancy Categories

The importance factor, I, for a seismic-isolated building shall be taken as 1.0, regardless of occupancy category.

153.4 Configuration Requirements

Each structure shall be designated as being regular or irregular on the basis of the structural configuration above the isolation system.

153.5 Selection of Lateral Response Procedure

Any seismic-isolated structure may be, and certain seismic-isolated structures defined below shall be, designed using the dynamic lateral response procedure of Section 155.

153.5.1 ***Static Analysis.*** The static lateral response procedure of Section 154 may be used for design of a seismic-isolated structure, provided:

1. The structure is located at least 15 kilometer (km) from all active faults.
2. The structure is located on a soil profile with a site factor of S_1 or S_2.
3. The structure is located in Seismic Zone 3 or 4.
4. The structure above the isolation interface is equal to or less than four stories, or 65 feet, in height.
5. The isolated period of the structure, T_I, is equal to or less than 3.0 seconds.
6. The isolated period of the structure, T_I, is greater than three times the elastic, fixed-base period of the structure above the isolation interface, as determined by Equation 105-5 of Section 105.2.2.2.
7. The structure above the isolation system is of regular configuration.
8. The isolation system is defined by all of the following attributes:
   a. The effective stiffness of the isolation system at the design displacement is greater than one third of the effective stiffness at 20 percent of the design displacement.
   b. The isolation system is capable of producing a restoring force, as specified in Section 157.2.4.
   c. The isolation system has force-deflection properties independent of the rate of loading.
   d. The isolation system has force-deflection properties independent of vertical load and bilateral load.

e. The isolation system does not limit maximum capable earthquake displacement to less than M_M times the total design displacement.

153.5.2 ***Dynamic Analysis.*** The dynamic lateral response procedure of Section 155 shall be used for design of seismic-isolated structures, as specified below:

153.5.2.1 ***Response Spectrum Analysis.*** Response spectrum analysis may be used for design of a seismic-isolated structure, provided:

1. The structure is located on a soil profile with a site factor of S_1, S_2, or S_3.
2. The isolation system is defined by all of the attributes specified in Section 153.5.1, Item 8.

153.5.2.2 ***Time-History Analysis.*** Time-history analysis may be used for design of any seismic-isolated structure and shall be used for design of all seismic-isolated structures not meeting the criteria of Section 153.5.2.1.

153.5.2.3 ***Site-Specific Design Spectra.*** Site-specific ground motion spectra of the design basis earthquake and the maximum capable earthquake, developed in accordance with Section 106.2, shall be used for design and analysis of all seismic-isolated structures as specified below:

1. The structure is located on a soil profile with a site factor of S_3 or S_4.
2. The structure is located within 15 km of an active fault.
3. The structure is located in Seismic Zone 1, 2A, or 2B.
4. The isolated period of the structure, T_I, is greater than 3.0 seconds.

154 UBC §1654 Static Lateral Response Procedure for Seismic-Isolated Structures

154.1 General

Except as provided in Section 155, every seismic-isolated structure, or portion thereof, shall be designed and constructed to resist minimum earthquake displacements and forces, as specified by this section and the applicable Requirements of Section 105.

154.2 Deformation Characteristics of the Isolation System

154.2.1 Minimum lateral earthquake design displacements and forces on seismic-isolated structures shall be based on the deformation characteristics of the isolation system.

154.2.2 The deformation characteristics of the isolation system shall explicitly include the effects of the wind-restraint system if such a system is used to meet the design Requirements of this document.

154.2.3 The deformation characteristics of the isolation system shall be based on properly substantiated tests performed in accordance with Section 161.

154.3 Minimum Lateral Displacements

154.3.1 ***Design Displacement.*** The isolation system shall be designed and constructed to withstand minimum lateral earthquake displacements that act in the direction of each of the main horizontal axes of the structure in accordance with the equation:

$$D = \frac{10ZNS_IT_I}{B}$$ **Eqn. 154-1**

See Table 154-1 for damping coefficient, *B;* and Table 154-2 for site coefficient, S_I.

154.3.2 ***Isolated Structure Period.*** The isolated-structure period, T_I, shall be determined using the deformational characteristics of the isolation system in accordance with the equation:

$$T_I = 2\pi\sqrt{\frac{W_I}{k_{min}g}}$$ **Eqn. 154-2**

154.3.3 ***Total Design Displacement.*** The total design displacement, D_T, of elements of the isolation system shall include additional displacement due to actual and accidental torsion calculated considering the spatial distribution of the lateral stiffness of the isolation system and the most disadvantageous location of mass eccentricity.

The total design displacement, D_T, of elements of an isolation system with uniform spatial distribution of lateral stiffness shall not be taken as less than that prescribed by the equation:

$$D_T = D\left[1 + y\frac{12e}{b^2 + d^2}\right]$$ **Eqn. 154-3**

The total design displacement, D_T, may be taken as less than the value prescribed by Equation 154-3, but not less than 1.1 times *D*, provided the isolation system is shown by calculation to be configured to resist torsion accordingly.

154.3.4 ***Total Maximum Displacement.*** The total maximum displacement, D_{TM}, required for verification of isolation system stability in the most critical direction of horizontal response shall be calculated in accordance with the equation:

$$D_{TM} = M_M D_T$$ **Eqn. 154-4**

154.4 Minimum Lateral Forces

154.4.1 ***Isolation System and Structural Elements at or Below the Isolation Interface.*** The isolation system, the foundation, and all structural elements at or below the isolation interface shall be designed and constructed to withstand a minimum lateral seismic force, V_b, using all of the appropriate requirements for a nonisolated structure, where:

$$V_b = \frac{k_{max}D}{1.5}$$ **Eqn. 154-5**

154.4.2 ***Structural Elements Above the Isolation System.*** The structure above the isolation system shall be designed and constructed to withstand a minimum shear force, V_s, using all of the appropriate requirements for a nonisolated structure where:

$$V_s = \frac{k_{max}D}{R_{wI}}$$ **Eqn. 154-6**

The R_{wI} factor shall be based on the type of lateral force resisting system used for the structure above the isolation system.

154.4.3 ***Limits on V_s.*** The value of V_s shall not be taken as less than the following:

1. The lateral seismic force required for a fixed-base structure of the same weight, W_I, and a period equal to the isolated period, T_I.
2. The base shear corresponding to the design wind load.
3. The lateral seismic force required to fully activate the isolation system (e.g., the yield level of a softening system, the ultimate capacity of a sacrificial wind-restraint system, or the static friction level of a sliding system).

154.5 Vertical Distribution of Force

Total force shall be distributed over the height of the structure above the isolation interface in accordance with the equation:

$$F_x = \frac{V_s w_x h_x}{\sum_{i=1}^{n} w_i h_i}$$ **Eqn. 154-7**

At each level designated as x, the force F_x shall be applied over the area of the building in accordance with the mass distribution at that level. Stresses in each structural element shall be calculated as the effect of force, F_x, applied at the appropriate levels above the base.

154.6 Drift Limits

The maximum interstory drift ratio of the structure above the isolation system shall not exceed $0.010/R_{wI}$ (Table 152-1).

155 UBC §1655 Dynamic Lateral Response Procedure for Seismic-Isolated Structures

155.1 General

As required by Section 153, every seismic-isolated structure, or portion thereof, shall be designed and constructed to resist earthquake displacements and forces, as specified in this section and the applicable Requirements of Section 106.

155.2 Isolation System and Structural Elements at or Below the Isolation Interface

155.2.1 The total design displacement of the isolation system shall not be taken as less than 90 percent of D_T, as specified by Section 154.3.3.

155.2.2 The total maximum displacement of the isolation system shall not be taken as less than 80 percent of D_{TM} as prescribed by Equation 154-4.

155.2.3 The design lateral shear force on the isolation system and structural elements below the isolation system shall not be taken as less than 90 percent of V_b as prescribed by Equation 154-5.

155.2.4 The limits of Items 1 and 2 above shall be evaluated using values of D_T and D_{TM} determined in accordance with Section 154.3, except that D' may be used in lieu of D, where D' is prescribed by the equation:

$$D' = \frac{D}{\sqrt{1 + (T/T_I)^2}}$$ **Eqn. 155-1**

where T is the elastic, fixed-base period of the structure above the isolation system, as determined by Equation 105-5 of Section 105.

155.3 Structural Elements Above the Isolation System

The design lateral shear force on the structure above the isolation system, if regular in configuration, shall not be taken as less than 80 percent of $k_{max}D/R_{wI}$, or less than the limits specified by Section 154.4.3.

> *Exception*: The design lateral shear force on the structure above the isolation system, if regular in configuration, may be taken as less than 80 percent of $k_{max}D/R_{wI}$, but not less than 60 percent of $k_{max}D/R_{wI}$, provided time-history analysis is used for design of the structure.

The design lateral shear force on the structure above the isolation system, if irregular in configuration, shall not be taken as less than $k_{max}D/R_{wI}$, or less than the limits specified by Section 154.4.3.

> *Exception*: The design lateral shear force on the structure above the isolation system, if irregular in configuration, may be taken as less than $k_{max}D/R_{wI}$, but

not less than 80 percent of $k_{max}D/R_{wI}$, provided time-history analysis is used for design of the structure.

155.4 Ground Motion

155.4.1 ***Design Spectra.*** Properly substantiated, site-specific spectra are required for design of all structures with an isolated period, T_I, greater than 3.0 seconds, or located on a soil type profile of S_3 or S_4, or located within 15 km of an active fault or located in Seismic Zone 1, 2A or 2B. Structures that do not require site-specific spectra and for which site-specific spectra have not been calculated shall be designed using spectra based on Figure 104-2.

A design spectrum shall be constructed for the design basis earthquake. This design spectrum shall not be taken as less than the normalized response spectrum given in Figure 104-2 for the appropriate soil type, scaled by the product ZN.

> ***Exception:*** If a site-specific spectrum is calculated for the design basis earthquake, then the design spectrum may be taken as less than 100 percent, but not less than 80 percent of the normalized response spectrum given in Figure 104-2 for the appropriate soil type, scaled by the product ZN.

A design spectrum shall be constructed for the maximum capable earthquake. This design spectrum shall not be taken as less than the normalized spectrum given in Figure 104-2 for the appropriate soil type, scaled by the product M_MZN. This design spectrum shall be used to determine the total maximum displacement and overturning forces for design and testing of the isolation system.

> ***Exception:*** If a site-specific spectrum is calculated for the maximum capable earthquake, then the design spectrum may be taken as less than 100 percent, but not less than 80 percent of the normalized response spectrum given in Figure 104-2 for the appropriate soil type, scaled by the product M_MZN.

155.4.2 ***Time Histories.*** Pairs of horizontal ground motion time-history components shall be selected from not less than three recorded events. For each pair of horizontal ground motion components, the square-root-sum-of-the-squares (SRSS) of the 5 percent damped spectrum of the scaled horizontal components shall be constructed. The motions shall be scaled such that the average value of the SRSS spectrum does not fall below 1.3 times the 5 percent-damped spectrum of the design basis earthquake (or maximum capable earthquake) by more than 10 percent in the period range of T_I, as determined by Equation 154-2, for periods from T_I minus 1.0 second to T_I plus 2.0 seconds.

The duration of the time histories shall be consistent with the magnitude and source characteristics of the design basis earthquake (or maximum capable earthquake).

Time histories developed for sites within 15 km of a major active fault shall incorporate near-fault phenomena.

155.5 Mathematical Model for Seismic-Isolated Structures

155.5.1 *General.* The mathematical models of the isolated structure, including the isolation system, the lateral force resisting system, and other structural elements, shall conform to Section 106.3 and to the Requirements of Sections 155.5.2 and 155.5.3 below.

155.5.2 *Isolation System*

155.5.2.1 The isolation system shall be modeled using deformational characteristics developed and verified by test in accordance with the Requirements of Section 154.2.

155.5.2.2 The isolation system shall be modeled with sufficient detail such that it will:

1. Account for the spatial distribution of isolator units.
2. Calculate translation, in both horizontal directions, and torsion of the structure above the isolation interface, considering the most disadvantageous location of mass eccentricity.
3. Assess overturning/uplift forces on individual isolator units.
4. Account for the effects of vertical load, bilateral load and/or the rate of loading if the force-deflection properties of the isolation system are dependent on one or more of these attributes.

155.5.3 *Isolated Structure*

155.5.3.1 ***Displacement.*** The maximum displacement of each floor and the total design displacement and total maximum displacement across the isolation system shall be calculated using a model of the isolated structure that incorporates the force-deflection characteristics of nonlinear elements of the isolation system and the lateral force resisting system.

1. Isolation systems with nonlinear elements include, but are not limited to, systems that do not meet the criteria of Section 153.5.1 Item 8.
2. Lateral force resisting systems with nonlinear elements include, but are not limited to, irregular structural systems designed for a lateral force less than $k_{max}D/R_{wI}$ and regular structural systems designed for a lateral force less than 80 percent of $k_{max}D/R_{wI}$.

155.5.3.2 ***Forces and Displacements in Key Elements.*** Design forces and displacements in key elements of the lateral force resisting system may be calculated using a linear elastic model of the isolated structure, provided:

1. Pseudo-elastic properties assumed for nonlinear isolation system components are based on the maximum effective stiffness of the isolation system.
2. All key elements of the lateral force resisting system are linear.

155.6 Description of Analysis Procedures

155.6.1 ***General.*** A response spectrum analysis or a time history analysis, or both, shall be performed in accordance with Sections 106.4 and 106.5 and the Requirements of this Section 155.6.

155.6.2 ***Input Earthquake.*** The design basis earthquake shall be used to calculate the total design displacement of the isolation system and the lateral forces and displacements of the isolated structure. The maximum capable earthquake shall be used to calculate the total maximum displacement of the isolation system.

155.6.3 *Response Spectrum Analysis*

1. Response spectrum analysis shall be performed using a damping value equal to the effective damping of the isolation system or 30 percent of critical, whichever is less.
2. Response spectrum analysis used to determine the total design displacement and the total maximum displacement shall include simultaneous excitation of the model by 100 percent of the most critical direction of ground motion and 30 percent of the ground motion on the orthogonal axis. The maximum displacement of the isolation system shall be calculated as the vectorial sum of the two orthogonal displacements.

155.6.4 *Time-History Analysis*

1. Time-history analysis shall be performed with at least three appropriate pair of horizontal time-history components, as defined in Section 155.4.2.
2. Each pair of time histories shall be applied simultaneously to the model, considering the most disadvantageous location of mass eccentricity. The maximum displacement of the isolation system shall be calculated from the vectorial sum of the two orthogonal components at each time step.
3. The parameter of interest shall be calculated for each time-history analysis. If three time-history analyses are performed, then the maximum response of the parameter of interest shall be used for design. If seven or more time-history analyses are performed, then the average value of the response parameter of interest may be used for design.

155.7 Design Lateral Force

155.7.1 ***Isolation System and Structural Elements at or Below the Isolation Interface.*** The isolation system, foundation, and all structural elements below the isolation interface shall be designed using all of the appropriate requirements for a nonisolated structure and the forces obtained from the dynamic analysis reduced by a factor of 1.5.

155.7.2 ***Structural Elements Above the Isolation System.*** Structural elements above the isolation system shall be designed using the appropriate requirements for a nonisolated structure and the forces obtained from the dynamic analysis reduced by a factor of R_{wI}. The R_{wI} factor shall be based on the type of lateral force resisting system used for the structure above the isolation system.

155.7.3 ***Scaling of Results.*** When the factored lateral shear force on structural elements, determined using either response spectrum or time-history analysis, is less than minimum levels prescribed by Sections 155.1 and 155.2, then all response parameters, including member forces and moments, shall be adjusted upward proportionally.

155.8 Drift Limits

Maximum interstory drift corresponding to the design lateral force, including displacement due to vertical deformation of the isolation system, shall not exceed the following limits:

1. The maximum interstory drift ratio of the structure above the isolation system, calculated by response spectrum analysis, shall not exceed $0.015/R_{wI}$.
2. The maximum interstory drift ratio of the structure above the isolation system, calculated by time-history analysis that considers the force-deflection characteristics of nonlinear elements of the lateral force resisting system, shall not exceed $0.020/R_{wI}$.
3. The secondary effects of the maximum capable earthquake lateral displacement of the structure above the isolation system, combined with gravity forces, shall be investigated if the interstory drift ratio exceeds $0.010/R_{wI}$.

156 UBC §1656 Lateral Load on Elements of Structures and Nonstructural Components Supported by Seismic-Isolated Structures

156.1 General

Parts or portions of an isolated structure, permanent nonstructural components and attachments to them, and attachments for permanent equipment shall be designed to resist seismic forces and displacements as prescribed by this section and the applicable Requirements of Section 107.

156.2 Forces and Displacements

156.2.1 ***Components at or Above the Isolation Interface.*** Elements of seismic-isolated structures and nonstructural components, or portions thereof at or above the isolation interface shall be designed to resist a total lateral seismic force equal to the maximum dynamic response of the element or component under consideration, divided by a factor of 1.5.

> *Exception*: Elements of seismic-isolated structures and nonstructural components, or portions thereof, may be designed to resist total lateral seismic force as prescribed by Equation 107-1 of Section 107.

156.2.2 ***Components That Cross the Isolation Interface.*** Elements of seismic-isolated structures and nonstructural components, or portions thereof, that cross the isolation interface shall be designed to withstand the total maximum displacement.

156.2.3 ***Components Below the Isolation Interface.*** Elements of seismic-isolated structures and nonstructural components, or portions thereof, below the isolation interface shall be designed and constructed in accordance with the Requirements of Section 107.

157 UBC §1657 Detailed Systems Requirements for Seismic-Isolated Structures

157.1 General

The isolation system and the structural system shall comply with the Requirements of Section 108 and the material Requirements of Chapters 2 through 6. In addition, the isolation system shall comply with the detailed system requirements of this section and the structural system shall comply with the detailed system requirements of this section and the applicable portions of Section 108.

157.2 Isolation System

157.2.1 ***Environmental Conditions.*** In addition to the requirements for vertical and lateral loads induced by wind and earthquake, the isolation system shall be designed with consideration given to other environmental conditions including aging effects, creep, fatigue, operating temperature, and exposure to moisture or damaging substances.

157.2.2 ***Wind Forces.*** Seismic-isolated structures shall resist design wind loads at all levels above the isolation interface in accordance with general wind design requirements. At the isolation interface, a wind restraint system shall be provided to limit lateral displacement in the isolation system to a value equal to that required between floors of the structure above the isolation interface.

157.2.3 ***Fire Resistance.*** Fire resistance for the isolation system shall meet that required for building columns, walls, or other structural elements.

157.2.4 ***Lateral Restoring Force.*** The isolation system shall be configured to produce a restoring force such that the lateral force at the total design displacement is at least $0.025W$ greater than the lateral force at 50 percent of the total design displacement.

> *Exception*: The isolation system need not be configured to produce a restoring force, as required above, provided the isolation system is capable of remaining stable under full vertical load and accommodating a total maximum displacement equal to the greater of either 3.0 times the total design displacement, or 36 $M_M Z N S_I$ inches.

157.2.5 ***Displacement Restraint.*** The isolation system may be configured to include a displacement restraint that limits lateral displacement due to the maximum capable earthquake to less than M_M times the total design displacement, provided that the seismic-isolated structure is designed in accordance with the following criteria when more stringent than the Requirements of Section 153:

1. Maximum capable earthquake response is calculated in accordance with the dynamic analysis Requirements of Section 155, explicitly considering the nonlinear characteristics of the isolation system and the structure above the isolation system.
2. The ultimate capacity of the isolation system and structural elements below the isolation system shall exceed the strength and displacement demands of the maximum capable earthquake.
3. The structure above the isolation system is checked for stability and ductility demand of the maximum capable earthquake.
4. The displacement restraint does not become effective at a displacement less than 0.75 times the total design displacement unless it is demonstrated by analysis that earlier engagement does not result in unsatisfactory performance.

157.2.6 ***Vertical Load Stability.*** Each element of the isolation system shall be designed to be stable under the full design vertical load ($1.2DL + 1.0LL + |E|$) at a horizontal displacement equal to the total maximum displacement. Vertical load due to earthquake, $|E|$, shall be based on peak response due to the maximum capable earthquake.

157.2.7 ***Overturning.*** The factor of safety against global structural overturning at the isolation interface shall not be less than 1.0 for required load combinations. All gravity and seismic loading conditions shall be investigated. Seismic forces for overturning calculations shall be based on the maximum capable earthquake, and W_I shall be used for the vertical restoring force.

Local uplift of individual elements is permitted, provided the resulting deflections do not cause overstress or instability of the isolator units or other building elements.

157.2.8 *Inspection and Replacement*

1. Access for inspection and replacement of all components of the isolation system shall be provided.
2. The architect or engineer of record, or a person designated by the architect or engineer of record, should complete a final series of inspections or observations of building separation areas and of components that cross the isolation interface prior to the issuance of the certificate of occupancy for the seismic-isolated building. Such inspections and observations should indicate that as-built conditions allow for free and unhindered displacement of the structure to maximum design levels and that all components that cross the isolation interface as installed are able to accommodate the stipulated displacements.
3. Seismic-isolated buildings should have a periodic monitoring inspection and maintenance program for the isolation system established by the architect or engineer responsible for the design of the system. The objective of such a program should be to ensure that all elements of the isolation system are able to perform to minimum design levels at all times.

4. Remodeling, repair, or retrofitting of the isolation system interface, including that of components that cross the isolation interface, should be performed under the direction of an architect or engineer licensed in the appropriate disciplines and experienced in the design and construction of seismic-isolated structures.
5. Horizontal displacement recording devices should be installed at the isolation interface in seismic-isolated buildings.

157.2.9 ***Quality Control.*** A quality control testing program for isolator units shall be established by the engineer responsible for the structural design.

157.3 Structural System

157.3.1 ***Horizontal Distribution of Force.*** A horizontal diaphragm or other structural elements shall provide continuity above the isolation interface and shall have adequate strength and ductility to transmit forces (due to nonuniform ground motion) from one part of the building to another.

157.3.2 ***Building Separations.*** Minimum separations between the isolated building and surrounding retaining walls or other fixed obstructions shall not be less than the total maximum displacement.

158 UBC §1658 Nonbuilding Seismic-Isolated Structures

Nonbuilding seismic-isolated structures shall be designed in accordance with the Requirements of Section 109 using design displacements and forces calculated in accordance with Sections 154 and 155.

159 UBC §1659 Foundations of Seismic-Isolated Structures

Foundations of seismic-isolated structures shall be designed and constructed in accordance with the Requirements of Chapter 3 of these Recommendations using design forces calculated in accordance with either Section 154 or Section 155.

160 UBC §1660 Design and Construction Review for Seismic-Isolated Structures

160.1 General

A design review of the isolation system and related test programs shall be performed by an independent engineering team, including persons licensed in the appropriate disciplines, experienced in seismic analysis methods, and the theory and application of seismic isolation.

160.2 Isolation System

Isolation system design review shall include, but not be limited to, the following:

1. Review of site-specific seismic criteria, including the development of site-specific spectra and ground motion time histories, and all other design criteria developed specifically for the project.
2. Review of the preliminary design, including determination of the total design displacement of the isolation system design displacement and lateral force design level.
3. Overview and observation of prototype testing (Section 161).
4. Review of the final design of the entire structural system and all supporting analyses.
5. Review of the isolation system quality control testing program (Section 157.2.9).

161 UBC §1661 Required Tests of Isolation System for Seismic-Isolated Structures

161.1 General

161.1.1 The deformation characteristics and damping values of the isolation system used in the design and analysis of seismic-isolated structures shall be based on tests of a selected sample of the components prior to construction.

161.1.2 The isolation system components to be tested shall include the wind restraint system, if such a system is used in the design.

161.1.3 The tests specified in Section 161 are for establishing and validating the design properties of the isolation system, and shall not be considered as satisfying the manufacturing quality control tests of Section 157.2.9.

161.2 Prototype Tests

161.2.1 ***General.*** Prototype tests shall be performed separately on two full-size specimens of each type and size of isolator unit in the isolation system. The test specimens shall include the wind restraint system, as well as individual isolator units, if such systems are used in the design. Specimens tested shall not be used for construction.

161.2.2 ***Record.*** For each cycle of tests, the force-deflection and hysteretic behavior of the test specimen shall be recorded.

161.2.3 ***Sequence and Cycles.*** The following sequence of tests shall be performed for the prescribed number of cycles at a vertical load equal to the average $DL + 0.5LL$ on all isolator units of a common type and size:

1. Twenty fully reversed cycles of loading at a lateral force corresponding to the design wind force.
2. Three fully reversed cycles of loading at each of the following increments of the total design displacement: 0.25, 0.50, 0.75, and 1.0.
3. Three fully reversed cycles at the total maximum displacement.

4. Fifteen S_I/B, but not less than 10, fully reversed cycles of loading at 1.0 times the total design displacement.

If an isolator unit is also a vertical load-carrying element, then Item 2 above shall be performed for two additional vertical load cases:

$$1.2DL + 0.5LL + |E|$$

$$0.8DL - |E|$$

In these tests, the combined vertical load shall be taken as the typical or average downward force on all isolator units of a common type and size.

161.2.4 ***Units Dependent on Loading Rates.*** If the force-deflection properties of the isolator units are dependent on the rate of loading, then each set of tests specified in Section 161.2.3 shall be performed dynamically at a frequency equal to the inverse of the effective period of the isolated structure.

If reduced-scale prototype specimens are used to quantify rate-dependent properties of isolators, the reduced-scale prototype specimens shall be of the same type and material and be manufactured with the same processes and quality as full-scale prototypes, and shall be tested at a frequency that represents full-scale prototype loading rates.

The force-deflection properties of an isolator unit shall be considered to be dependent on the rate of loading if there is greater than a plus or minus 10 percent difference in the effective stiffness at the design displacement when tested at a frequency equal to the inverse of the effective period of the isolated structure, and when tested at any frequency in the range of 0.1 to 2.0 times the inverse of the effective period of the isolated structure.

161.2.5 ***Units Dependent on Bilateral Load.*** If the force-deflection properties of the isolator units are dependent on bilateral load, then the tests specified in Sections 161.2.3 and 161.2.4 shall be augmented to include bilateral load at increments of the total design displacement 0.25 and 1.0, 0.50 and 1.0, 0.75 and 1.0, and 1.0 and 1.0.

> ***Exception***: If reduced-scale prototype specimens are used to quantify bilateral-load-dependent properties, then such scaled specimens shall be of the same type and material, and manufactured with the same processes and quality as full-scale prototypes.

The force-deflection properties of an isolator unit shall be considered to be dependent on bilateral load, if the bilateral and unilateral force-deflection properties have greater than a plus or minus 10 percent difference in effective stiffness at the design displacement.

161.2.6 ***Downward Vertical Load.*** Isolator units that carry vertical load shall be statically tested for maximum and minimum downward vertical load at total maximum displacement. In these tests, the combined vertical loads of $1.2DL + 1.0LL + |E|$ shall be taken as the maximum downward force, and the combined vertical load of $0.8DL - |E|$ shall be taken as the minimum downward force, on any one isolator unit of a common type and size.

161.2.7 ***Sacrificial Wind Restraint Systems.*** If a sacrificial wind-restraint system is to be employed, then the ultimate capacity of said system shall be established by test.

161.2.8 ***Testing Similar Units.*** Prototype tests are not required if an isolator unit is of similar size, of the same type and material, and manufactured with the same processes and quality control procedures as a prototype isolator unit previously tested using the specified sequence of tests.

161.3 Determination of Force-Deflection Characteristics

161.3.1 The force-deflection characteristics of the isolation system shall be based on the cyclic load test results for each fully reversed cycle of loading.

161.3.2 The effective stiffness of an isolator unit shall be calculated for each cycle of loading as follows:

$$k_{eff} = \frac{F^{+} - F^{-}}{\Delta^{+} - \Delta^{-}} \qquad \textbf{Eqn. 161-1}$$

where F^{+} and F^{-} are the maximum positive and maximum negative forces,. respectively, and Δ^{+} and Δ^{-} are the maximum positive and maximum negative test displacements, respectively.

1. If the minimum effective stiffness is to be determined, then F^{+}_{min} and F^{-}_{min} shall be used in the equation.
2. If the maximum effective stiffness is to be determined, then F^{+}_{max} and F^{-}_{max} shall be used in the equation.

161.4 System Adequacy

Performance of the test specimens shall be assessed as adequate if the following conditions are satisfied:

161.4.1 The force-deflection plots of all tests specified in Section 161.2 have a positive incremental force-carrying capacity.

161.4.2 For each increment of test displacement specified in Section 161.2.3 Item 2, and for each vertical load case specified in Section 161.2.3:

1. There is no greater than a plus or minus 10 percent difference between the effective stiffness at each of the three cycles of testing and the average value of effective stiffness for each test specimen.
2. There is no greater than a plus or minus 10 percent difference in the average value of effective stiffness of the two test specimens of a common type and size of the isolator unit over the required three cycles of testing.

161.4.3 For each specimen, there is no greater than a plus or minus 20 percent change in the initial effective stiffness of each test specimen over the $15S_I/B$, but not less than 10, cycles of testing specified in Section 161.2.3, Item 4.

161.4.4 For each specimen, there is no greater than a 20 percent decrease in the initial effective damping over the $15S_I/B$, but not less than 10, cycles of testing specified in Section 161.2.3, Item 4.

161.4.5 All specimens of vertical load-carrying elements of the isolation system remain stable at the total maximum displacement for static load, as prescribed in Section 161.2.6.

161.5 Design Properties of the Isolation System

161.5.1 ***Effective Stiffness.*** The minimum and maximum effective stiffnesses of the isolation system shall be determined as follows:

1. The value of k_{min} shall be based on the minimum effective stiffnesses of individual isolator units, as established by the cyclic tests of Section 161.2.3, Item 2 at a displacement amplitude equal to the design displacement.
2. The value of k_{max} shall be based on the maximum effective stiffnesses of individual isolator units, as established by the cyclic tests of Section 161.2.3, Item 2 at a displacement amplitude equal to the design displacement.
3. For isolator units that are found by the tests of Sections 161.2.3, 161.2.4, and 161.2.5 to have force-deflection characteristics that vary with vertical load, rate of loading or bilateral load, respectively, the value of k_{max} shall be increased and the value of k_{min} shall be decreased, as necessary, to bound the effects of measured variation in effective stiffness.

161.5.2 ***Effective Damping.*** Effective damping β of the isolation system, expressed as the ratio to critical damping, shall be calculated as:

$$\beta = \frac{1}{2\pi}\left[\frac{Total\ Area}{k_{eff}D^2}\right] \qquad \textbf{Eqn. 161-2}$$

where *Total Area* shall be taken as the sum of the areas of the hysteresis loops of all isolator units, and the hysteresis loop area of each isolator unit shall be taken as the minimum area of the three hysteresis loops established by the cyclic tests iterated in Section 161.2.3 Item 2 at a displacement amplitude equal to the design displacement.

Table 104-1. *Seismic Zone Factor Z (UBC Table 16-I)*

Zone*	1	2	3	4
Z	†	†	0.3	0.4

*The zone shall be determined from the seismic zone map in Figure 104-1.
†Not used in California.

Table 104-2. *Site Coefficients*[1]

Type	Description	S Factor
S_1	A soil profile with either: 1. A rock-like material characterized by a shear-wave velocity greater than 2,500 feet per second, or by other suitable means of classification or: 2. Medium stiff to stiff or medium dense to dense soil conditions where soil depth is less than 200 feet	1.0
S_2	A soil profile with predominantly medium dense to dense or medium stiff to stiff soil conditions, where soil depth exceeds 200 feet	1.2
S_3	A soil profile containing more than 20 feet of soft to medium stiff clay but not more than 40 feet of soft clay	1.5
S_4	A soil profile characterized by a shear wave velocity of less than 500 feet per second, and containing more than 40 feet of soft clay	2.0

[1] The site factor shall be established from properly substantiated geotechnical data. In locations where soil properties are not known in sufficient detail to determine the soil profile type, soil profile S_3 shall be used. Soil profile S_4 need not be assumed unless the Building Official determines that soil profile S_4 may be present at the site, or in the event that soil profile S_4 is established by geotechnical data.

Table 104-3. Occupancy Categories (similar to UBC Table 16-K)

Occupancy Category	Occupancy or Function of Structure	Seismic Importance Factor, *I*	Seismic[1] Importance Factor, *Ip*
I. Essential Facilities[2]	**Group I, Division 1.** Occupancies having surgery and emergency treatment areas. Fire and police stations. Garages and shelters for emergency vehicles and emergency aircraft. Structures and shelters in emergency preparedness centers. Aviation control towers. Structures and equipment in government communication centers and other facilities required for emergency response. Standby power-generating equipment for Category I facilities. Tanks or other structures containing housing or supporting water or other fire-suppression material or equipment required for the protection of Category I, II, or III structures.	1.25	1.50
II. Hazardous Facilities[2]	**Group H, Divisions 1, 2, 6, and 7.** Occupancies and structures therein housing or supporting toxic or explosive chemicals or substances. Nonbuilding structures housing, supporting, or containing quantities of toxic or explosive substances which, if contained within a building, would cause that building to be classified as a Group H, Division 1, 2 or 7 Occupancy.	1.25	1.50
III. Special Occupancy Structures[2,3]	**Group A, Divisions 1, 2 and 2.1 Occupancies.** Buildings housing Group E, Divisions 1 and 3 occupancies with a capacity greater than 300 students. Buildings housing Group B Occupancies used for college or adult education with a capacity greater than 500 students. Group I, Divisions 1 and 2 occupancies with 50 or more resident incapacitated patients, but not included in Category I. **Group I, Division 3 Occupancies.** All structures with an occupancy greater than 5,000 persons. Structures and equipment in power-generating stations, and other public utility facilities not included in Category I or Category II above, and required to maintain continuous operation	1.00	1.00
IV. Standard Occupancy Structures[3]	All structures housing occupancies or having functions not listed in Categories I, II, or III, and Group U Occupancy towers.	1.00	1.00
V. Miscellaneous	Group U occupancies	1.00	1.00

1. The limitation of *Ip* for panel connections in Section 108.2.4 shall be 1.0 for the entire connector.
2. Structural observation requirements are given in Chapter 2. Special review of the design and construction of structures classified as Occupancy Categories I, II, and III is required in Seismic Zones 2, 3, and 4. See Section 201.
3. For anchorage of machinery and equipment required for life safety systems, the value of I_p shall be taken as 1.5.

Table 104-4. *Vertical Structural Irregularities (UBC Table 16-L)*

Irregularity Type	Definition	Reference Section
A. Stiffness Irregularity (soft story)	A soft story is one in which the lateral stiffness is less than 70 percent of that in the story immediately above, or less than 80 percent of the average stiffnesses of the three stories above.	104.8.3.2
B. Weight (mass) Irregularity	Mass irregularity shall be considered to exist where the tributary mass of any story is more than 150 percent of the tributary mass of an adjacent story. A roof that is lighter than the floor below need not be considered.	104.8.3.2
C. Vertical Geometric Irregularity	Vertical geometric irregularity shall be considered to exist where the horizontal dimension of the lateral force resisting system in any story is more than 130 percent of that in an adjacent story. One-story penthouses need not be considered.	104.8.3.2
D. In-plane Discontinuity in Vertical Lateral Force Resisting Elements	An in-plane offset of the lateral load resisting elements greater than the length of those elements.	105.7.1.2
E. Discontinuity in Capacity (weak story)	A weak story is one in which the story strength is less than 80 percent of that of the strength of the story above. The story strength is the total strength of all seismic resisting elements sharing the story shear for the direction under consideration.	104.9.1

Table 104-5. *Plan Structural Irregularities (UBC Table 16-M)*

Irregularity Type	Definition	Reference Section
A. Torsional Irregularity (to be considered when diaphragms are not flexible)	Torsional irregularity shall be considered to exist when the maximum story drift, computed (including accidental torsion) at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts of the two ends of the structure.	105.6.4 108.1.3 108.2.9.6
B. Re-entrant Corners	Plan configurations of a structure and its lateral force resisting system contain re-entrant corners, where both projections of the structure beyond a re-entrant corner are greater than 15 percent of the plan dimension of the structure in the given direction.	108.2.9.6 108.2.9.7
C. Diaphragm Discontinuity	Diaphragms with abrupt discontinuities or variations in stiffness, including those having cutout or open areas greater than 50 percent of the gross enclosed area of the diaphragm or changes in effective stiffness of more than 50 percent from one story to the next.	108.2.9.6
D. Out-of-Plane Offsets	Discontinuities in a lateral force path, such as out-of-plane offsets of the vertical elements.	107.7.1.2 108.2.9.6
E. Nonparallel Systems	The vertical lateral load resisting elements are not parallel to, or symmetric about, the major orthogonal axes of the lateral force resisting system.	108.1.3

Table 104-6. *Structural Systems (UBC Table 16-N)*

Basic Structural System (Fn1)	Lateral Force Resisting System	R_w (Fn2)	H (Fn3)
A. Bearing Wall	1. Light framed walls with shear panels		
	a. Unbraced (cantilevered) parapets	8	65
	b. All other light framed walls	6	65
	2. Shear walls		
	a. Concrete	6	160
	b. Masonry	6	160
	3. Light steel framed bearing walls with tension-only bracing	4	65
	4. Braced frames where bracing carries gravity loads		
	a. Steel	6	160
	b. Concrete[4]	4	—
	c. Heavy timber	4	65
B. Building Frame	1. Steel eccentric braced frame (EBF)	10	240
	2. Light Framed walls with shear panels		
	a. Wood structural panel walls for structures three stories or less	9	65
	b. All other light framed walls	7	65
	3. Shear walls		
	a. Concrete	8	240
	b. Masonry	8	160
	4. Concentric braced frames		
	a. Steel	8	160
	b. Concrete[4]	8	—
	c. Heavy timber	8	65
	5. Special concentrically braced frames		
	a. Steel	9	240
C. Moment Resisting Frame	1. Special moment resisting frames (SMRF)		
	a. Steel	12	NL[5]
	b. Concrete	12	NL
	2. Concrete intermediate moment resisting frames (IMRF)[6]	7	—
	3. Masonry moment resisting wall frames (MMRWF)	9	160
	4. Ordinary moment resisting frames (OMRF)		
	a. Steel	6	160
	b. Concrete[4]	5	—

Table 104-6. (Continued) *Structural Systems (UBC Table 16-N)*

Basic Structural System (Fn1)	**Lateral Force Resisting System**	R_w (Fn2)	**H** (Fn3)
D. Dual System	1. Shear walls		
	a. Concrete with SMRF	12	NL
	b. Concrete with concrete IMRF[6]	9	—
	c. Masonry with SMRF	8	160
	d. Masonry with concrete IMRF[4]	7	—
	2. Steel EBF with steel SMRF	12	NL
	3. Concentric braced frames		
	a. Steel with steel SMRF	10	NL
	b. Concrete with concrete SMRF	9	—
	c. Concrete with concrete IMRF[4]	6	—
	4. Special concentrically braced frames		
	a. Steel with steel SMRF	11	NL
	b. Steel with steel OMRF	6	160
E. Cantilevered Column	1. Cantilevered column elements[6]	3	35
F. Undefined	See Section 104.9.2	—	—

1. Basic structural systems are defined in Section 104.6.
2. See Section 105.3 for combination of structural system.
3. H=height limit applicable to Seismic Zones 3 and 4 (see Section 104.7 for exceptions).
4. Prohibited in Seismic Zones 3 and 4.
5. NL=no limit.
6. Prohibited in Seismic Zones 3 and 4, except as permitted in Section 109.2

Table 104-7. *Horizontal Force Factor C_p (UBC Table 16-O)*

Item	Elements of Structures, Nonstructural Components, and Equipment[1]	Value of C_p
I. Elements of Structure	1. Walls, including the following:	
	a. Unbraced (cantilevered) parapets	2.00
	b. Exterior wall above the ground floor and parapets braced above their centers of gravity[2]	0.75
	c. All interior bearing and nonbearing walls	0.75
	2. Penthouses (except where framed by an extension of the structural frame)	0.75
	3. Connections for prefabricated structural elements other than walls[3]	0.75
	4. Diaphragms[4]	—
II. Nonstructural Components	1. Exterior and interior ornamentations and appendages	2.00
	2. Chimneys, stacks, and trussed towers supported on or projecting above the roof:	
	a. Cantilevered or and laterally braced or guyed to the structural frame at a point below their centers of mass	2.00
	b. Unbraced cantilever in which height above roof line exceeds distance to edge of structure	2.00
	c. Laterally braced or guyed to the structural frame at or above their centers of mass	0.75
	3. Signs and billboards	2.00
	4. Storage racks (include two-thirds contents) over 6 feet tall[5]	0.75
	5. Permanent floor-supported cabinets and book stacks more than 5 feet in height (include contents)[6]	0.75
	6. Suspended ceilings and light fixtures[3,7,8,9]	0.75
	7. Access floor systems[3,6,10]	0.75
	8. Masonry or concrete fences over 6 feet high	0.75
	9. Partitions	0.75
III. Equipment	1. Tanks and vessels (include contents), including support systems	0.75
	2. Electrical, mechanical, and plumbing equipment and associated conduit, and ductwork and piping[6,11,12,13,14,15,16,17,18,19]	0.75
	3. Any equipment supported on or projecting as an unbraced cantilever above the roof more than one half its total height	2.00
	4. Anchorage of emergency power supply systems and essential communications equipment. Anchorage and support systems for battery racks and fuel tanks necessary for operation of emergency equipment[20,21]	0.75
	5. Temporary containers with flammable or hazardous materials[22]	0.75

1. See Section 107.2.4 for items supported at or below grade.
2. See Section 108.2.4 and 108.2.8 for concrete and masonry walls and Section 107.2 for connections for panel connectors.
3. Applies to Seismic Zones 2, 3, and 4 only.
4. See Section 108.2.9.

5. In lieu of the tabulated values, steel storage racks may be designed using the provisions of Section 109, or in accordance with UBC Chapter 22, Division VI.
6. Only anchorage or restraints need be designed.
7. Ceiling weight shall include all light fixtures and other equipment or partitions laterally supported by the ceiling. For purposes of determining the seismic force, a ceiling weight of not less than 4 pounds per square foot (0.19 *kN/m2*) shall be used.
8. Ceilings constructed of lath and plaster or gypsum board screw or nail attached to suspended members that support a ceiling at one level and extending from wall to wall need not be analyzed, provided the walls are not over 50 feet (15,240 mm) apart.
9. Light fixtures and mechanical services installed in metal suspension systems for acoustical tile and lay-in panel ceilings shall be independently supported from the structures above, as specified in UBC Standard 25-2, Part III.
10. W_p for access floor systems shall be the dead load of the access floor system plus 25 percent of the floor live load plus a 10 psf (0.48 *kN/m2*) partition load allowance.
11. Equipment includes, but is not limited to, boilers, chillers, heat exchangers, pumps, air-handling units, cooling towers, control panels, motors, switchgear, transformers, and life-safety equipment. It shall include major conduit, ducting, and piping serving such machinery and equipment and fire sprinkler systems. See Section 107.2.3 for additional requirements for determining C_p for flexible or flexibly mounted equipment.
12. Seismic restraints may be omitted from piping and duct supports if all of the following conditions are satisfied:
    a. Lateral motion of the piping or duct will not cause damaging impact with other systems.
    b. The piping or duct is made of ductile material with ductile connections.
    c. Lateral motion of the piping or duct does not cause impact of fragile appurtenances (e.g. sprinkler heads) with any other equipment, piping, or structural member.
    d. Lateral motion of the piping or duct does not cause loss of system vertical support.
    e. Rod-hung supports of less than 12 inches in length have top connections that cannot develop moments.
    f. Support members cantilevered up from the floor are checked for stability.
    g. Vertical load-carrying supports are designed to carry at least three times the dead load plus contents tributary to the support.
13. Seismic restraints may be omitted from electrical raceways such as cable trays, conduit, and bus ducts if all of the following conditions are satisfied:
    a. Lateral motion of the raceway will not cause damaging impact with other systems.
    b. Lateral motion of the raceway does not cause loss of system vertical support.
    c. Rod-hung supports of less than 12 inches in length have top connections that cannot develop moments.
    d. Support members cantilevered up from the floor are checked for stability.
    e. Vertical load-carrying supports are designed to carry at least three times the dead weight tributary to the support.
14. Piping, ducts, and electrical raceways that must function following an earthquake, and which span between different buildings or structural systems, must be sufficiently flexible to withstand relative motion of support points assuming out-of-phase motions. Displacement of each structure or system shall be taken as $3R_w/8$ times the code-calculated displacements, unless actual expected displacements are estimated by analysis.
15. Vibration isolators supporting equipment must be designed for lateral loads or restrained from displacing laterally by other means. For design of these restraints, the product of $C_f C_g$ shall be greater than or equal to 2.0. Restraint must also be provided limiting vertical displacement, such that lateral restraints do not become disengaged.
16. Equipment anchorage shall not be designed such that lateral loads are resisted entirely by friction (e.g., friction clips).

17. Expansion anchors required to resist seismic loads in tension shall not be used where operational vibrating loads are significant.
18. The design of elevators and elevator machinery should conform to the requirements of the American Society of Mechanical Engineers (ASME), ASME A17.1, Safety Code for Elevators and Escalators, 1973.
19. The vertical load-carrying supports for piping, ducting, and other distribution systems suspended overhead shall be designed to carry at least three times the dead weight tributary to the support.
20. Movement of components within electrical cabinets, rack- and skid-mounted equipment, and portions of skid-mounted electromechanical equipment that may cause damage to other components by displacing, must be restricted by attachment to anchored equipment or support frames.
21. Batteries on racks shall be restrained to ensure that batteries will not fall off the rack in an earthquake. Batteries shall also be prevented from rocking and colliding with each other or damaging bus bars.
22. Seismic restraints may include straps, chains, bolts, barriers, or other mechanisms that prevent sliding, falling, and breach of containment of flammable and/or toxic materials. Friction forces may not be used to resist lateral loads in these restraints unless positive uplift restraint is provided that will ensure that the friction forces act continuously.

Table 104-8. *Factors for Nonbuilding Structures (UBC Table 16-P)*

Structure Type	R_W
1. Vessels, including tanks and pressurized spheres, on braced or unbraced legs	3
2. Cast-in-place concrete silos and chimneys that have walls contiguous to and with the foundation	5
3. Distributed mass cantilever structures such as stacks, chimneys, silos, and skirt-supported vertical vessels	4
4. Trussed towers (freestanding or guyed), guyed stacks, and chimneys	4
5. Inverted pendulum-type structures	3
6. Cooling towers	5
7. Bins and hoppers on braced or unbraced legs	4
8. Storage racks	5
9. Signs and billboards	5
10. Amusement structures and monuments	3
11. All other self-supporting structures not otherwise covered	4

Table 152-1. Structural Systems Above the Isolation Interface

Basic Structural System(Fn1)	Lateral Force Resisting System	R_{wI} (Fn2)	H (Fn3)
A. Bearing Wall	1. Light framed walls with shear panels		
	a. Unbraced (cantilevered) parapets	2.6	65
	b. All other light framed walls	2.2	65
	2. Shear walls		
	a. Concrete	2.6	160
	b. Masonry	2.6	160
	3. Light steel framed bearing walls with tension-only bracing	1.6	65
	4. Braced frames where bracing carries gravity loads		
	a. Steel	1.8	160
	b. Concrete[4]	1.5	
	c. Heavy timber	1.5	65
B. Building Frame	1. Steel eccentric braced frame (EBF)	3.0	240
	2. Light framed walls with shear panels		65
	a. Wood structural panel walls for structures three stories or less	2.6	65
	b. All other light framed walls	2.2	
	3. Shear walls		
	a. Concrete	3.0	240
	b. Masonry	3.0	160
	4. Concentric braced frames		
	a. Steel	2.2	160
	b. Concrete[4]	2.6	
	c. Heavy timber	2.6	65
C. Moment Resisting Frame	1. Special moment resisting frames (SMRF)		
	a. Steel	3.0	NL[5]
	b. Concrete	3.0	NL
	2. Concrete intermediate moment resisting frames (IMRF)[6]	2.2	
	3. Masonry moment resisting wall frames (MMRWF)		
	4. Ordinary moment resisting frames		
	a. Steel	1.8	160
	b. Concrete[4]	1.5	

Table 152-1. (Continued) *Structural Systems Above the Isolation Interface*

Basic Structural System (Fn1)	Lateral Force Resisting System	R_{wI} (Fn2)	H (Fn3)
D. Dual System	1. Shear walls		
	a. Concrete with SMRF	3.0	NL
	b. Concrete with concrete IMRF[6]	2.6	160
	c. Masonry with SMRF	3.0	160
	d. Masonry with concrete IMRF[4]	2.6	
	2. Steel EBF with steel SMRF	2.6	NL
	3. Concentric braced frames		
	a. Steel with steel SMRF	2.2	NL
	b. Concrete with concrete SMRF	2.2	
	c. Concrete with concrete IMRF[4]	1.8	
E. Undefined	See Section 104.9.2	—	—

1. Basic structural systems are defined in Section 104.6.
2. See Section 105.3 for combination of structural system.
3. H=height limit applicable to Seismic Zones 3 and 4 (see Section 104.7 for exceptions).
4. Prohibited in Seismic Zones 3 and 4.
5. NL=no limit.
6. Prohibited in Seismic Zones 3 and 4, except as permitted in Section 109.2.

Table 152-2. *Maximum Capable Earthquake Response Coefficient, M_M*

	Response Region[1,2]	
Seismic Zone	Constant Acceleration	Constant Velocity
4	1.25	1.25
3	1.33	1.50

1. Except for construction of design spectra, as required in Section 155.4, the value of M_M shall be that specified for constant velocity. The value of M_M for sites not located in Seismic Zone 3 or 4 shall be taken as the ratio of maximum capable earthquake and design basis earthquake spectra at the effective period, T_I, of the isolated structure.
2. For construction of design spectra, as required in Section 155.4, the value of M_M specified for constant acceleration shall apply to all periods from 0 second to the transition period (i.e., 0.4 second for Soil Type S_1, 0.6 second for Soil Type S_2 and 0.9 second for Soil Type S_3), and the value of M_M specified for constant velocity shall apply to all periods greater than the transition period. The product $M_M ZN$ in the constant velocity region need not exceed the product $M_M ZN$ at the transition period.

Table 152-3. Near-Field Coefficient, N

Closest Distance to Active Fault (d_F)	Maximum Capable Earthquake Magnitude of Active Fault (M_{MCE})[1,2]		
	$M_{MCE} \geq 8.0$	$M_{MCE} = 7.0$	$M_{MCE} \leq 6.0$
$d_F \geq 15$ km	1.0	1.0	1.0
$d_F = 10$ km	1.2	1.0	1.0
$d_f \leq 5$ km	1.5	1.3	1.1

1. Location and maximum capable earthquake magnitude, M_{MCE}, of active faults shall be established for properly substantiated geotechnical data (e.g., most recent mapping of active faults by the United States Geological Survey or the California Division of Mines and Geology).
2. The near-field site response coefficient shall be based on linear interpolation of values of the closest distance, d_F, and maximum capable magnitude, M_{MCE}, other than those given.

Table 154-1. Damping Coefficient, B

Effective Damping β (%), (percentage of critical)[1,2]	B
≤ 2%	0.8
5%	1.0
10%	1.2
20%	1.5
30%	1.7
40%	1.9
≥ 50%	2.0

1. The damping coefficient shall be based on the effective damping of the isolation system determined in accordance with the Requirements of Section 161.5.
2. The damping coefficient shall be based on linear interpolation for effective damping values other than those given.

Table 154-2. Site Coefficients for Seismic-Isolated Structures

Soil Profile Type[1]	S_I Factor
S_1	1.0
S_2	1.4
S_3	2.3
S_4	2.7

1. See Table 104-2 of these Recommendations for a description of soil profile types and related requirements.

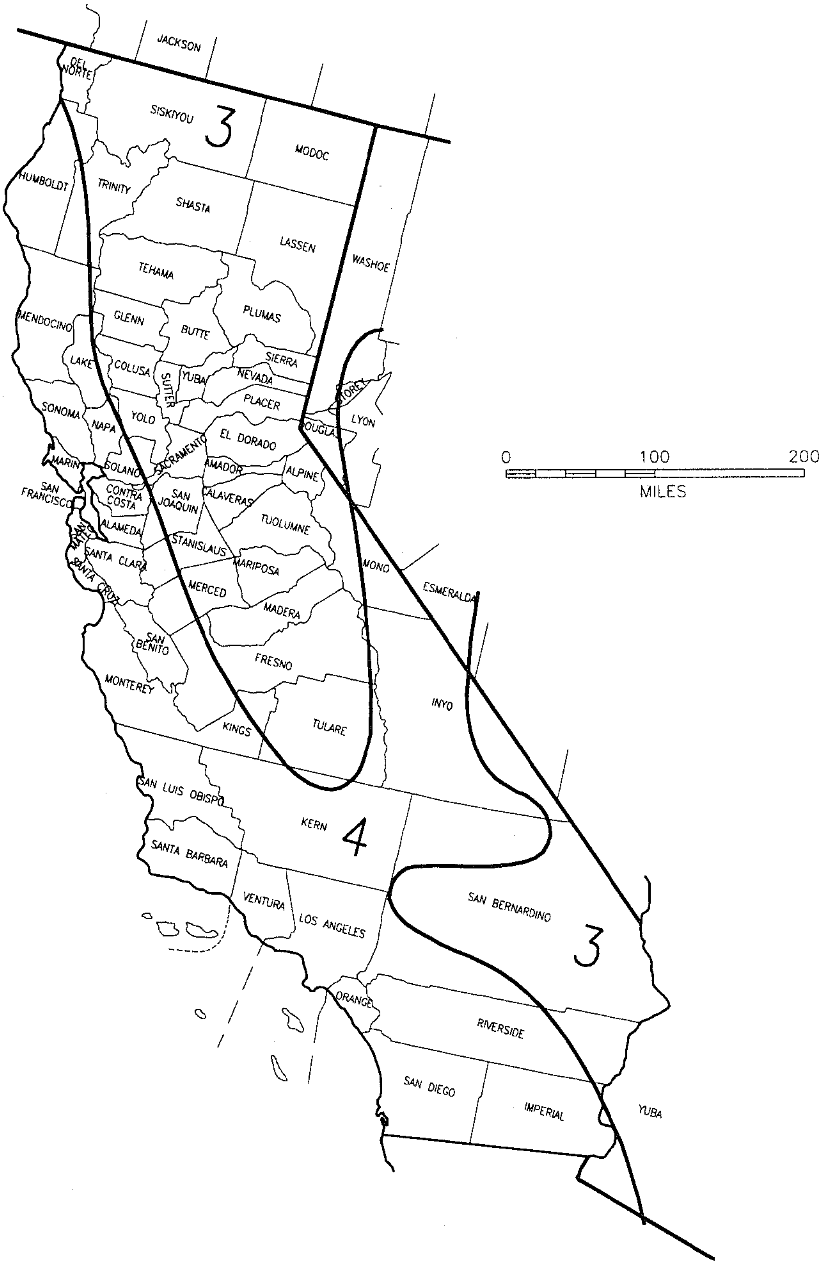


Figure 104-1. *Seismic zone map for California*

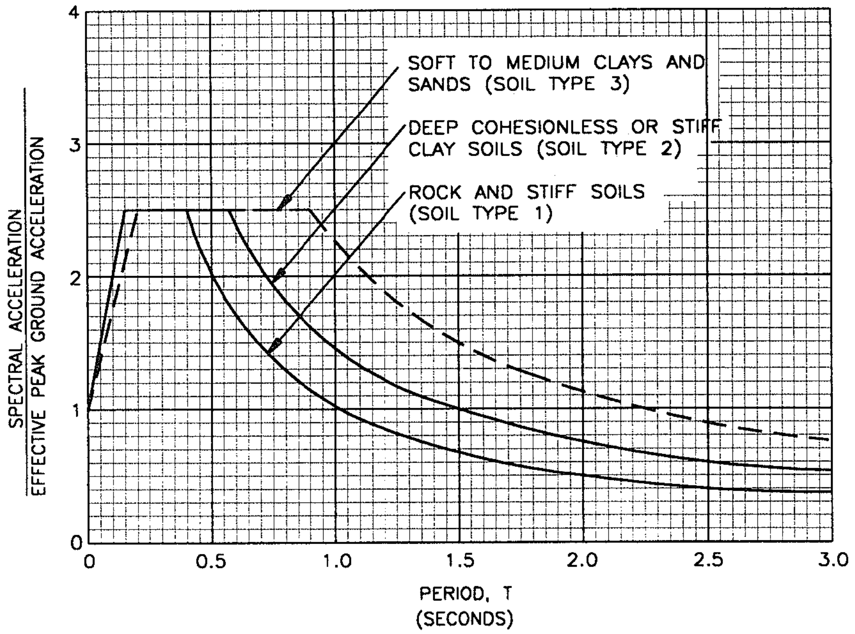


Figure 104-2. Normalized response spectra shapes

CHAPTER 2

Structural Tests and Inspections

201 General Requirements

Structural inspections and tests for elements of the seismic force resisting system and nonstructural components and equipment anchorage subject to the requirements of Chapter 1 shall conform to the requirements of the 1994 edition of UBC, except as modified by Requirements of this chapter.

All buildings shall have design review and review of the construction during the progress of the construction. Special review of the design and construction of buildings housing Occupancy Categories I, II, or III in Seismic Zones 2, 3, or 4 is required.

Where complex or innovative building systems are involved it is advisable, and may be required by the jurisdiction in which the proposed structure will be located, to use project design peer review.

202 Recommended Modifications to 1994 UBC

Items listed below are modifications to the 1994 UBC requirements, as approved by the SEAOC Seismology Committee. These items may or may not be adopted into future editions of the UBC.

202.1 Special Inspections

202.1.1 *Modify:* ***UBC §1701.5, Item 11 Piling, Drilled Piers, and Caissons.***
Add at end of existing paragraph as follows:

Piling, drilled piers, and caissons: verify that the soil at the site is that assumed in the design, when required by the Quality Assurance Plan.

202.1.2 *Add new item:* ***UBC§1701.5, Item 16 Wood***

For diaphragms and shear walls with boundary or panel edge nail spacing less than 4 inches on centers: the inspection shall verify nail size and spacing and that nail heads do not penetrate the sheathing surface.

For connections for diaphragms and shear walls which provide a load path for seismic force resistance, including anchor bolts, tie downs, concrete or masonry wall anchors and drag strut splices; the inspection shall verify connector sizes and proper installation including end and edge clearances, condition of member at connectors and tightening of connectors.

202.2 Structural Observation

Modify: ***UBC §1702 Structural Observation.*** *Delete existing text and replace as follows:*

Structural observation shall be provided by the design engineer(s) of record or their authorized representatives. Structural observation shall consist of site visits at intervals appropriate to the stage of construction to observe construction in progress and review testing and inspection reports for general compliance with the construction documents relating to the structural work and the nonstructural components and equipment anchorage. A report shall be made at the completion of the structural work, the nonstructural components and anchorage regarding the observations that have been made.

202.3 Design Review

Add new section: ***UBC §1705 Design Review***

The special design review required by Section 201 for buildings housing Occupancy Categories I, II or III in Seismic Zones 2, 3, or 4 shall be by an independent structural engineer licensed in the state in which the building is constructed. For other Occupancy Categories, the design review required by Section 201 shall be by an independent engineer licensed in the state in which the building is constructed.

202.4 Project Design Peer Review

Add new section: ***UBC §1706 Project Design Peer Review***

For complex or innovative building systems, the owner may wish to involve a project design peer reviewer or review panel. The jurisdiction where the building is to be built may require that the owner invoke project design peer review. The peer review process should start in the conceptual stage and continue through the end of construction document production and approval by the jurisdiction.

The project design peer review shall be an independent and objective technical review to examine the basic concepts, objectives, and criteria proposed for the project. The design reviewer(s) shall be independent from the design team, experienced in the type of project being reviewed, and led by a structural engineer registered in the state where the project is to be constructed.

202.5 Quality Assurance

Add new section: ***UBC §1707 Quality Assurance***

A quality assurance program shall be prepared by the design engineer and shall be made a part of the construction documents. The program shall specify the type and extent of materials testing to be performed and the extent of field and shop inspection required to obtain compliance with the design documents. This quality assurance program shall be executed by an independent laboratory or inspector accepted by the design engineer and approved by the Building Official. This program is in addition to the quality control administered by the Contractor for his or her own operations.

CHAPTER 3

Foundations

301 General Requirements

The design and construction of foundation systems shall conform to the requirements of the 1994 edition of the UBC, except as modified by the Requirements of this chapter.

302 Recommended Modifications to 1994 UBC

Items listed below are modifications to the 1994 UBC requirements, as approved by the SEAOC Seismology Committee. These items may or may not be adopted into future editions of the UBC.

302.1 Allowable Foundation and Lateral Pressures

Modify: ***UBC §1805 Allowable Foundation and Lateral Pressures.*** *Add new paragraph at beginning of section to read as follows:*

The capacity of the soil in bearing, or the capacity of the soil interface between pile, pier, or caisson and the soil, shall be sufficient to support the structure with all prescribed loads, other than earthquake forces, taking due account of the settlement that the structure is capable of withstanding. For load combinations including earthquake, the soil capacity must be sufficient to resist loads at acceptable deformations considering both the cyclic nature of loading and the dynamic properties of the soil. Bearing capacity for total loads, including earthquake forces, may be increased by more than 33 percent of the capacity for dead plus live loads, if substantiated by structural and geotechnical data.

302.2 Requirements for Piles, Pile Caps, and Grade Beams in Seismic Zones 3 and 4

302.2.1 *Modify:* ***UBC §1809.5.1 General Requirements.*** *Add the following sentence:*

Piles shall be designed for flexure when displaced by earthquake motions.

302.2.2 *Add new subsection:* ***UBC §1809.5.2.4 Additional Requirements for Piles***

1. Splices of pile segments shall develop the full strength of the pile.
2. Concrete piles shall be designed as columns in accordance with UBC §1921.4 using appropriate strength reduction factors, ϕ, from UBC §1909. The percentage of longitudinal reinforcement in non-prestressed concrete piles shall exceed the lesser of one-half of one percent of the gross section area or one percent of the core section area. For piles in competent soils, the volume of transverse reinforcement for confinement of concrete in potential plastic hinge regions may be reduced by multiplying the values in UBC §1921.4 by the following:

   $$0.5 + 1.4\frac{P_u}{f'_c A_g} \qquad \textbf{Eqn. 302-1}$$

   Friction or bond shall not be considered to provide load transfer between concrete and steel of metal cased concrete piles.
3. Steel piles shall be designed as columns in accordance with UBC §§2211.5.2, 2211.5.3, and 2211.7.3.
4. Interaction effects of axial and flexure forces for precast, prestressed concrete piles shall conform to UBC §1918.11.1.
5. The strength of concrete piles in axial compression and flexure shall be multiplied by a factor of 0.70 when h_c/h is less than 0.70.
6. Load factors for reinforced concrete design shall conform with UBC §1921.2.7.

302.2.3 *Add new subsection:* ***UBC §1809.5.3 Pile-Soil-Structure Interaction***

1. Pile moments, shears and lateral deflections used for design shall be established considering the nonlinear interaction of the shaft and soil, as recommended by a geotechnical engineer. Where the ratio of the depth of embedment of the pile-to-the-pile diameter or width is less than or equal to 6, the pile may be assumed to be rigid.
2. Pile group effects from soil on lateral pile capacity shall be considered where pile center-to-center spacing in the direction of lateral force is less than 8 pile diameters. Pile group effects on vertical capacity shall be considered where pile center-to-center spacing is less than 3 pile diameters.
3. In Seismic Zones 3 and 4, flexure due to seismic soil deformations shall be considered in the design of piles in soil profile categories S3 and S4, and in sites with a potential for soil liquefaction strength loss. Such piles shall comply with the minimum special detailing requirements for members subjected to flexure and axial load as specified in UBC §1910 and §1921 for concrete piles, and UBC §2211 for steel piles, unless it is shown through analysis that lesser requirements are sufficient.

4. The mass of the nonembedded portion of grade beams and pile caps shall be considered as part of the inertial force on the piles.
5. Lateral passive soil resistance at vertical projected surfaces of pile caps, grade beams, walls, and piles may be combined to resist inertial forces from the superstructure. The use of frictional resistance, resulting from soil pressure normal to structural member surfaces, shall be neglected where soil settlement or structure/soil deformations could result in loss of soil contact.

302.2.4 *Add new subsection:* ***UBC §1809.6 Special Requirements for Pile Caps and Grade Beams***

1. Grade beams shall be designed as beams in accordance with UBC §1921.3 and §1921.4. When grade beams have the capacity to resist the forces from the load combination in Section 105.7.2, they need not conform to UBC §1921.3 and §1921.4.
2. Design of anchorage of piles into the pile cap shall consider combined effect of uplift forces and fixity to the pile cap. Anchorage shall develop a minimum of 25 percent of the allowable tensile value of the pile. For piles required to resist uplift forces or provide rotational restraint, anchorage into the pile cap shall be capable of developing, at a minimum, the lesser of the following:
    a. The tensile strength of the longitudinal reinforcement in a concrete pile or the tensile strength of a steel pile.
    b. 1.3 times the pile uplift capacity in the soil.
3. Where the vertical lateral force resisting elements are columns, the grade beam or pile cap flexural strengths shall exceed the column flexural strength.
4. The distance from the face of the pile away from the cap or grade beam edges shall not be less than 6 inches.

CHAPTER 4

Reinforced Concrete

401 General Requirements

Design and construction of reinforced concrete components that are subject to seismically induced forces or deformations shall conform to the requirements of the 1994 UBC, except as modified by the Requirements of this chapter.

402 Recommended Modifications to 1994 UBC

Items listed below are modifications to the 1994 UBC requirements, as approved by the SEAOC Seismology Committee. These items may or may not be adopted into future editions of the UBC.

402.1 Details of Reinforcement

402.1.1 *Modify*: ***UBC §1907.1.3, Item 4 Stirrups and Ties.*** *To read as follows:*

For stirrup and tie hooks in Seismic Zones 3 and 4, refer to the hoop and crosstie provisions of §1921.1.

402.1.2 *Modify*: ***UBC §1907.10.5.6 Hooks and Column Ties.*** *To read as follows:*

Column ties shall have hooks as specified in §1907.1.3.

402.2 Strength Reduction Factors

402.2.1 *Modify*: ***UBC §1909.3.4.1.*** *To read as follows:*

The shear strength reduction factor shall be 0.6 for the design of walls, topping slabs used as diaphragms over precast concrete members and structural framing members, with the exception of joints, if their nominal shear strength is less than the shear corresponding to the development of their flexural strength. The nominal flexural strength shall be determined corresponding to the most critical factored axial loads including earthquake effects. The shear strength reduction factor for joints shall be 0.85.

402.2.2 *Modify:* ***UBC §1909.3.4.2.*** *Delete entire section and replace as follows (Ed. Note: the section deleted is not related to the replacement, see Commentary):*

Reinforcement used for diaphragm chords or collectors shall be designed using a strength reduction factor of 0.60.

402.3 Limits of Section

Modify: ***UBC §1910.8.4 Limits of Section.*** *To read as follows:*

For a compression member with a cross-section larger than required by considerations of loading, it shall be permitted to base the minimum reinforcement and design strength on a reduced effective area A_g not less than one-half the total area. This provision shall not apply in Seismic Zones 3 and 4.

402.4 Circular Columns

Add new subsection: ***UBC §1911.5.6.9 Circular Columns***

For circular columns, the area used to compute V_c shall be $0.8A_g$. The shear strength V_s provided by the circular transverse reinforcing shall be computed by:

$$V_s = \frac{\pi A_b f_{yh} D'}{2s}$$ **Eqn. 402-1**

where A_b is the area of the hoop or spiral bar of yield strength f_{yh} with pitch s and hoop or spiral diameter D'.

402.5 Reinforced Concrete Structures Resisting Forces Induced by Earthquake Motions

Add notations: ***UBC §1921.0 Notations***

h = overall thickness of member, inches (mm)

l_u = unsupported length of compression member (see UBC §1910.11.1.1)

402.6 Flexural Members of Frames

Modify: ***UBC §1921.3.1 Scope.*** *To read as follows:*

Requirements of this section apply to frame members: 1) resisting earthquake-induced forces and 2) proportioned primarily to resist flexure. Members that resist gravity loads by axial compression shall conform to UBC §1921.4.

402.7 Frames Members Subjected to Bending and Axial Loads

402.7.1 *Modify:* ***UBC §1921.4.1 Scope.*** *To read as follows:*

The requirements of UBC §1921.4 apply to frame members 1) resisting earthquake-induced forces and 2) having a factored axial force exceeding ($A_g f_c'/10$), and 3) to all members of the system resisting gravity loads by axial compression. These frame members shall also satisfy the following conditions:

402.7.2 *Modify*: ***UBC §1921.4.3.2 Splice Locations.*** *To read as follows:* Welded splices and mechanical connections shall conform to UBC §1921.2.6.1. Lap splices are permitted only within the center half of the member length. They shall be proportioned as Class A tension lap splices in accordance with UBC §1912 and shall have transverse reinforcement over the lap splice length in accordance with §1921.3.2.3.

402.8 Shear Strength

Modify: ***UBC §1921.5.3.1 Shear Strength.*** *Replace last paragraph in this section with the following:*

where A_j is the effective cross-sectional area within the joint in a plane parallel to the plane of the reinforcement generating shear in the joint. The joint depth shall be the overall depth of the column. Where a beam frames into a support of larger width, effective width of the joint shall not exceed the smaller of:

1. Beam width plus overall depth of joint.
2. Twice the smaller perpendicular distance from the longitudinal axis of the beam to the column side.

402.9 Design of Shear Walls for Flexural and Axial Loads

402.9.1 *Modify*: ***UBC §1921.6.5.2.*** *To read as follows:*

The effective flange widths to be used in the design of I-, L-, C-, or T-sections shall not be assumed to extend further from the face of the web than 1) one-half the distance to adjacent shear wall web, or 2) 15 percent of the total wall height.

402.9.2 *Modify formula*: ***UBC §1921.6.5.4, Item 3.*** *To read as follows:*

3. $V_u \le 3A_{cv}\sqrt{f'_c}$ and $M_u/(V_u l_w) \le 3.0$ For SI: $V_u \le 0.25A_{cv}\sqrt{f'_c}$

402.9.3 *Modify*: ***UBC §1921.6.5.5, Item 2 Determination of Compressive Strains.*** *Delete Item 2 and replace as follows:*

2. For shear walls in which the flexural limit state response is governed by yielding at the base of the wall, compressive strains at wall edges may be approximated as follows:

Determine the total curvature demand (ϕ_t) as given in formula (21-9):

$$\phi_t = \frac{\Delta_i}{(h_w - l_p/2)l_p} + \phi_y \qquad \text{UBC Eqn. 21-9}$$

Assume a linear variation of strain with depth and compute the maximum compressive strain as follows:

$$\varepsilon_c = \phi_t c'_u$$

where:

ε_c = compressive strain

c'_u = neutral axis depth corresponding to the displacement Δ_m. The neutral axis depth c'_u may be approximated as the neutral axis depth at nominal flexural strength, determined in accordance with Section 1910.2, accounting for the axial load P'_u and accounting for all developed vertical reinforcement.

P'_u = $1.2D + 0.5L + E$

l_p = vertical length of the plastic hinge above the critical section and which shall be established on the basis of substantiated test data or may alternatively be taken as $l_p = 0.5 l_w$

Δ_s = elastic design displacement at the top of the wall using the effective moment of inertia over the height of the wall and the code-specified design seismic forces

Δ_i = inelastic deflection at top of wall, equals $\Delta_m - \Delta_y$

Δ_m = total elastic plus inelastic deflection at the top of the wall equal to $R_w \Delta_s$

Δ_y = equivalent yield displacement at top of wall and may be taken as: $\Delta_y = \Delta_s(M'_n / M_s)$ where M_s is the moment corresponding to the design seismic forces and M'_n is the nominal flexural strength corresponding to P'_u .

ϕ_y = sectional curvature determined by a strain compatibility analysis at first yield in tension of the extreme fiber longitudinal reinforcement in the critical section, which may be taken as: $\phi_y = 0.003 / 1_w$

402.9.4 *Modify*: ***UBC §1921.6.5.6, Item 1.1 Shear Wall Boundary Zone Detail Requirements.*** *To read as follows:*

1.1 All portions of the boundary zone shall have a thickness of 1/16 times the clear height of the wall or greater.

402.10 Boundaries of Structural Diaphragms

Add new subsection: ***UBC §1921.6.6.3 Boundaries of Structural Diaphragms***
Reinforcement for chords and collectors at splices and anchorage zones shall have a minimum spacing of 3 bar diameters, but not less than 1.5 inches (40 mm) and a minimum concrete cover of 2.5 bar diameters, but not less than 2 inches (50 mm) and shall have transverse reinforcement as specified by UBC §1911.5.5.3, except as required in UBC §1921.6.2.3.

402.11 Coupling Beams

Replace: ***UBC §1921.6.9*** *Delete entire section and replace with new sections 1921.6.9.1 through 1921.6.9.5 as follows:-*

UBC §1921.6.9 Coupling Beams

UBC §1921.6.9.1 For coupling beams with $l_n/d \geq 4$, the design shall conform to the requirements of UBC §§1921.2 and 1921.3. It shall be permitted to waive the requirements of UBC §1921.3.1.3 and 1921.3.1.4 if it can be shown by analysis that lateral stability is adequate or if alternate means of maintaining lateral stability is provided.

UBC §1921.6.9.2 Coupling beams with $(l_n/d < 4)$ shall be permitted to be reinforced with two intersecting groups of symmetrical diagonal bars. Coupling beams with $(l_n/d < 4)$ and with factored shear force V_u exceeding $4\sqrt{f'_c}b_w d$ shall be reinforced with two intersecting groups of symmetrical diagonal bars. Each group shall consist of a minimum of four bars assembled in a core with lateral dimension of each side not less than $(b_w/2)$ or 4 inches (102 mm). The design shear strength, ϕV_n, of these coupling beams shall be determined by:

$$\phi V_n = 2\phi f_y A_{vd} \sin\alpha \leq 10\phi\sqrt{f'_c}\,b_w d \qquad \textbf{Eqn. 402-2}$$

where: α = the angle between the diagonal reinforcement and the longitudinal axis

A_{vd} = the total area of reinforcement of each group of diagonal bars

ϕ = 0.85

> ***Exception:*** The design of coupling beams need not comply with the requirements for diagonal reinforcement if it can be shown that failure of the coupling beams will not impair the vertical load carrying capacity of the structure, the egress from the structure, or the integrity of nonstructural components and connections or produce other unacceptable effects. The analysis shall take into account the effects of the failure of the coupling beams on foundation rotation and overall system displacements. Design strength of coupling beams assumed to be part of the seismic force resisting system shall not be reduced below the values otherwise specified.

UBC §1921.6.9.3 Each group of placed bars shall be enclosed in transverse reinforcement conforming to UBC §1921.4.4.1 through 1921.4.4.3. For the purpose of computing A_g, as per UBC Formula 10-5 and 21-3, the minimum cover, as specified in UBC §1907.7, shall be assumed over each group of diagonally placed reinforcing bars.

UBC §1921.6.9.4 Reinforcement parallel and transverse to the longitudinal axis shall be provided and, as a minimum, shall conform to UBC §1910.5, §1911.8.9, and §1911.8.10.

UBC §1921.6.9.5 Contribution of the diagonal reinforcement to nominal flexural strength of the coupling beams shall be considered.

402.12 Minimum Thickness of Diaphragms

Modify: ***UBC §1921.6.11*** *To read as follows:*
Diaphragms used to resist prescribed lateral forces shall not be less than 2 inches (50 mm) thick. Topping slabs placed over precast floor and roof elements shall not be less than 3 inches (75 mm) or $6d_b$ thick, where d_b is the diameter of the largest reinforcement in the topping slab.

402.13 Frame Members Not Part of the Lateral Force Resisting System

Replace: ***UBC §1921.7*** *Delete entire section and replace as follows:*

UBC §1921.7 Frame Members Not Part of the Lateral Force Resisting System. Frame members assumed not to contribute to lateral resistance shall be detailed according to UBC §1921.7.2 or 1921.7.3, depending on the magnitude of moments induced in those members when subjected to $3(R_w/8)$ times the displacements resulting from the specified lateral forces in conformance with the requirements of UBC §1631.2.4. When induced moments under lateral displacements are not calculated, UBC §1921.7.3 shall apply.

UBC §1921.7.2 When the induced moments and shears under the lateral displacements of UBC §1921.7 combined with the factored gravity moments and shears do not exceed the design moment and shear strength of the frame member, the following conditions shall be satisfied. For this purpose, the load combinations $(1.4D+1.4L)$ and $(0.9D)$ shall be used.

UBC §1921.7.2.1 Members with factored axial gravity forces not exceeding $(A_g f'_c/10)$ shall satisfy UBC §1921.3.2.1. Stirrups shall be placed at not more than d/2 throughout the length of the member.

UBC §1921.7.2.2 Members with factored axial gravity forces exceeding $(A_g f'_c/10)$, but not exceeding $0.3P_o$ shall satisfy UBC §1921.4.3; §1921.4.4.1, Item 3; and §1921.4.4.3. Design shear strength shall not be less than the shear associated with the development of nominal moment strengths of the member at each end of the clear span. The maximum longitudinal spacing of ties shall be (s_o) for the full column height. The spacing, (s_o), shall not be more than: 1) Six diameters of the smallest longitudinal bar enclosed; 2) Sixteen tie-bar diameters; 3) One-half the least cross sectional dimension of the column; and 4) Six inches.

UBC §1921.7.2.3 Members with factored gravity axial forces exceeding $0.3P_o$ shall satisfy UBC §§1921.4.4 and 1921.4.5.

UBC §1921.7.3 When the induced moments under lateral displacements of UBC §1921.7.1 exceed the design moment strength of the frame member, or where induced moments are not calculated, the following conditions shall be satisfied:

UBC §1921.7.3.1 Materials shall satisfy UBC §§1921.2.4, 1921.2.5, and 1921.2.6.

UBC §1921.7.3.2 Members with factored gravity axial forces not exceeding $(A_g f'_c / 10)$ shall satisfy §§1921.3.2.1 and 1921.3.4. Stirrups shall be placed at not more than $d/2$ throughout the length of the member.

UBC §1921.7.3.3 Members with factored gravity axial forces exceeding $(A_g f'_c / 10)$ shall satisfy UBC §§1921.4.4, 1921.4.5, 1921.5.2.1.

CHAPTER 5

Lightweight Metals

501 General Requirements

The methodology for the design and construction of lightweight metal members and their fastenings for resistance to seismic forces has not yet been examined in detail by the SEAOC Seismology Committee. Consequently, until research is conducted to determine design procedures and detail requirements, it is recommended that caution be used in assigning stress levels and determining detail configurations.

CHAPTER 6
Reinforced Masonry

601 General Requirements

The design and construction of reinforced masonry components that resist seismic forces shall conform to the requirements of the 1994 edition of the UBC, except as modified by the Requirements of this chapter.

602 Recommended Modifications to 1994 UBC

Items listed below are modifications to the 1994 UBC requirements, as approved by the SEAOC Seismology Committee. These items may or may not be adopted into future editions of the UBC.

602.1 Standards of Quality

Modify: ***UBC §2102.2, Item 10 Reinforcement***
The use of plain bars in reinforced masonry construction is prohibited, except for joint reinforcement.

602.2 General Design Requirements

602.2.1 *Modify:* ***UBC §2106.1.12.4, Item 2.1 Reinforcement.*** *Add the following paragraph:*

Longitudinal masonry joint reinforcement may be used in reinforced hollow unit masonry to fulfill minimum reinforcement ratios, but it shall not be considered in the determination of the shear strength of the member.

602.2.2 *Modify:* ***UBC §2106.3.2 Plain Bars.*** *Delete text and replace with the following:*

The use of plain bars in reinforced masonry construction is prohibited, except for joint reinforcement.

602.3 Design of Reinforced Masonry

602.3.1 *Delete:* ***UBC §2107.2.2 Reinforcement.*** *Delete entire section.*

602.3.2 *Delete:* ***UBC §2107.2.12 Lap Splice Increases.*** *Delete entire section.*

602.4 Strength Design of Masonry

Modify: ***UBC §2108.2.2.6 Development.*** *Revise UBC equation 8-21 to read as follows:*

$$l_{de} = \frac{0.15 d_b^2 f_y}{K\sqrt{f_{m'}}}$$

CHAPTER 7
Structural Steel

701 General Requirements

The design and construction of steel framing and components that resist seismic forces shall conform to requirements of the 1994 edition of the UBC, except as modified by the Requirements of this chapter.

702 Recommended Modifications to 1994 UBC

Items listed below are modifications to the 1994 UBC requirements, as approved by the SEAOC Seismology Committee. These items may or may not be adopted into future editions of the UBC.

702.1 *Delete:* ***UBC §2211.7.1.2*** *Delete entirely.*

702.2 *Modify:* ***UBC §2211.7.1.3*** **Connection Strength.** *Change title and revise as follows:*

Connection configurations utilizing welds or high strength bolts shall demonstrate, by approved cyclic test results or calculation, the ability to sustain inelastic rotations and develop the strength criteria in UBC §2211.7.1.1 considering the effects of steel overstrength and strain hardening.

CHAPTER 8
Wood

801 General Requirements

The design and construction of wood members and their fastenings that resist seismic forces shall conform to the requirements of the 1994 edition of the UBC, except as modified by the Requirements of this chapter.

802 Recommended Modifications to 1994 UBC

Items listed below are modifications to the 1994 UBC requirements, as approved by the SEAOC Seismology Committee. These items may or may not be adopted into future editions of the UBC.

802.1 Safe Lateral Strength

802.1.1 *Add new subsection:* ***UBC §2311.3.1.1 Toenails and Slant Nailing***
Toenails and slant nailing shall not be used to transfer seismic forces in excess of 150 pounds per foot (2188 N/m) from diaphragms to shear walls, drag struts (collectors), or other elements, nor from shear walls to other elements.

802.1.2 *Add new subsection:* ***UBC §2314.3.1.2 Diaphragms***
Size and shape of horizontal diaphragms and vertical diaphragms (shear walls) shall be limited as set forth in Table 802-1. The height of vertical diaphragms shall be defined as:

1. The maximum height from foundation to bottom of diaphragm framing above. *or:*
2. The maximum height from top of diaphragm to bottom of diaphragm framing above.

The width of a vertical diaphragm shall be defined as the width of sheathing.

Where vertical diaphragms with openings are designed for force transfer around the openings, the limitations of Table 802-1 shall apply to the overall vertical diaphragm, including openings and to each wall pier at the side of an opening. The height of a wall pier shall be defined as the height of a pier at the side of an opening.

The width of a wall pier shall be defined as the sheathed width of a pier at the side of an opening.

802.2 Wood Shear Walls and Diaphragms in Seismic Zones 3 and 4

802.2.1 *Modify:* ***UBC §2314.5.2 Framing.*** *Add to existing text as follows:* Wood framing members shall be at least 2-inch nominal thickness.

802.2.2 *Modify:* ***UBC §2314.5.3 Wood Structural Panels.*** *Add to existing text as follows:*

Wood shear walls shall not be used to resist seismic forces in concrete or masonry buildings over one story in height.

> ***Exception:*** Wood shear walls may be used to resist seismic forces from concrete or masonry walls in buildings two stories in height with story heights not exceeding 12 feet, provided the allowable shear in all wood shear walls is limited to 50 percent of that otherwise permitted.

802.2.3 *Modify:* ***UBC §2314.5.4 Heavy Wood Panels.*** *Add to existing text as follows:*

Framing members for heavy wood panels shall be of at least 4-inch nominal width. Board decking shall be of at least 2-inch nominal thickness; decking shall be at least 1-1/8 inches thick, or 1-1/8-inch-thick plywood.

802.2.4 *Add new subsection:* ***UBC §2314.5.6 Sill Plates and Framing Members***

In Seismic Zones 3 and 4, for wood shear walls where design shear values exceed 350 pounds per foot (5.11 N/mm), foundation sill plates and wood framing members receiving edge nailing from abutting panels shall be of at least a single 3-inch (76 mm) nominal thickness. Nails shall be staggered.

802.2.5 *Add new subsection:* ***UBC §2314.5.7 Foundation Plates or Sills***

In Seismic Zone 4, foundation anchor bolts shall have steel plate washers of not less than 2 inches by 2 inches by 3/16 inch. The anchor bolt center line shall be located not more than 2 inches from the sheathed face of the foundation sill plate. Alternately, other sill plate anchorage devices, demonstrated to provide equal resistance to foundation sill plate cross-grain tension (flexural) failure, shall be permitted.

802.3 Conventional Light Frame Construction

802.3.1 *Modify:* ***UBC §2326.1 General.*** *Add to existing text as follows:*

Lateral bracing for conventional light frame construction shall be as indicated in Table 802-2 below. For a description of Methods 1 through 8, see UBC §2326.11.3.

Method 1 is not permitted in Seismic Zones 2B, 3, and 4. For cripple wall bracing, see Table 803-3. For Methods 2, 3, 4, 6, 7, and 8, each braced wall panel must be at least 48 inches (1219 mm) in length, covering a minimum of three studs that are spaced not more than 24 inches (1610 mm) on center.

802.3.2 *Modify:* ***UBC §2326.11.5 Cripple Walls.*** *Add to existing text as follows:*

Cripple walls that have a stud height exceeding 14 inches (356 mm) shall be considered to be first-story walls for the purposes of determining the bracing required by Table 802-2. See Table 802-3 for minimum cripple wall bracing. Solid blocking or wood structural panel sheathing shall be used to brace cripple walls that have a stud height of 14 inches (356 mm) or less.

Table 802-1. *Maximum Diaphragm Dimension Ratios (UBC Table 23-I-I)*

Material	Horizontal Diaphragms Max. Span-Width Ratios	Vertical Diaphragms Max. Height/Width Ratios
Diagonal sheathing, conventional	3:1	1:1[1]
Diagonal sheathing, special	4:1	2:1[2]
Wood structural panels and particle board, nailed all edges	4:1	2:1[2]
Wood structural panels and particle board, blocking omitted at intermediate joints	4:1	____[3]

1. In Seismic Zones 0, 1, and 2, the maximum ratio may be 2:1.
2. In Seismic Zones 0, 1, and 2, the maximum ratio may be 3.5:1.
3. Not permitted.

Table 802-2. *Wall Bracing for Seismic Zones 2B, 3, and 4 (UBC Table 23-I-W)*

	Construction Method								**Braced Panel Location and Length**
Condition	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	
One-story, top of two-story, or three-story		x	x	x	x	x	x	x	Each end and not more than 25 feet (7620 mm) on center
First story of two-story, or second of three-story		x	x	x	x	x	x	x	Each end and not more than 25 feet (7620 mm) on center, but not less than 25% of building length
First story of three-story		x	x	x	x	x	x	x	Each end and not more than 25 feet (7620 mm) on center, but not less than 40% of building length

"x" indicates system permitted; see also the footnotes to UBC Table 23-I-W.

Table 802-3. *Cripple Wall Bracing*

Seismic Zone	**Condition**	**Minimum Amount of Cripple Wall Bracing[1,2] x 25.4 for mm**
4	One-story above cripple wall	3/8" wood structural panel with 8d at 6"/12"[3] nailing on 60% of building length
	Two-story above cripple wall	3/8" wood structural panel with 8d at 4"/12" nailing on 50% of building length *or:* 3/8" wood structural panel with 8d at 6"/12" nailing on 75% of building length
3	One-story above cripple wall	3/8" wood structural panel with 8d at 6"/12" nailing on 40% of building length
0,1,2	One-story above cripple wall	3/8" wood structural panel with 8d at 6"/12" nailing on 30% of building length
0,1,2,3	Two-story above cripple wall	3/8" wood structural panel with 8d at 4"/12" nailing on 30% of building length *or:* 3/8" wood structural panel with 8d at 6"/12" nailing on 60% of building length

1. Braced panel length shall be at least two times the height of the cripple wall, but not less than 48 inches (1220 mm).
2. All braced panels along a wall shall be nearly equal in length and shall be nearly equally spaced along the length of the building.
3. 6"/12" = 6" edge nailing, 12" field nailing.

CHAPTER 9

Other Materials

901 General Requirements

Materials that have had structural properties established may be designed by either allowable stress methods or by strength methods. Approved structural methods may be used, as these may be familiar to designers of similar or analogous materials already in use. For assemblies to be designed by strength methods, all components shall have had ultimate strength values established, or the entire assembly shall have been tested for ultimate strength properties.

902 Recommended Modifications to 1994 UBC

None.

903 Other Sheathing

Refer to UBC sections on fiberboard shear wall sheathing and Chapter 23 on non-wood wall covering.

Commentary

CHAPTER 1

General Requirements for Design and Construction of Earthquake-Resistive Structures

C101 General

C101.1 Scope and Objectives

This Commentary is given to provide the users of this Blue Book with an interpretation of the intent of these Requirements, the bases for these provisions and their limitations, and suggested procedures for cases and conditions outside the limits of regular practice. Since data, knowledge, and/or experience may not be adequate in some areas, this Commentary expresses concerns and alternate points of view, where appropriate.

These Requirements provide minimum standards for use in building design regulation to maintain public safety in the extreme ground shaking likely to occur during an earthquake. These Requirements are intended primarily to safeguard against major failures and loss of life, not to limit damage, maintain function, or provide for easy repair. It is emphasized that the purpose of these recommended design procedures is to provide buildings that are *expected* to meet this life safety objective.

The specified design forces given herein are based on the assumption that a significant amount of inelastic behavior may take place in the structure subjected to significant ground shaking. As a result, these design forces and related elastic deformations are much lower than those that would be required if the structure were to remain elastic. For a given structural system, the design Requirements outlined herein are intended to provide for the necessary inelastic behavior. Representations of the element force levels and deformations in the fully responding inelastic structure are given as appropriate multiples of the values that would be determined by linear elastic analysis of the structure using the design forces specified in this document.

C101.1.1 ***Damage Levels.*** Structures designed in conformance with these Requirements should, in general, be able to:

1. Resist a minor level of earthquake ground motion without damage.

2. Resist a moderate level of earthquake ground motion without structural damage, but possibly experience some nonstructural damage.
3. Resist a major level of earthquake ground motion—of an intensity equal to the strongest earthquake, either experienced or forecast, for the building site—without collapse, but possibly with some structural as well as nonstructural damage.

It is expected that structural damage, even in a major design level earthquake, will be limited to a repairable level for most structures that meet these Requirements. In some instances, damage may not be economically repairable. The level of damage depends upon a number of factors, including the intensity and duration of ground shaking, structure configuration, type of lateral force resisting system, materials used in the construction, and construction workmanship.

C101.1.2 ***No Guarantee.*** Conformance to these Requirements does not constitute any kind of guarantee or assurance that significant structural damage will not occur in the event of design level earthquake ground motion. To fulfill the life safety objective of these Requirements, there are requirements that provide for structural stability in the event of extreme structural deformations and requirements that protect the vertical load-carrying system from fracture or buckling at these extreme states. While damage to the primary structural system may be either negligible or significant, repairable or virtually irreparable, it is reasonable to expect that a well-planned and constructed structure will not collapse in a major earthquake. The protection of life is reasonably provided, but not with complete assurance.

Conformance to these Requirements will not limit or prevent damage due to earth movements, including earth slides such as those that occurred in Anchorage, Alaska, or due to soil liquefaction such as occurred in Niigata, Japan. These Requirements are intended to provide minimum required resistance to earthquake ground shaking.

C101.1.3 ***Minimum Design Criteria.*** These Requirements provide minimum design criteria in specific categories stated in broad general terms. It is the responsibility of the engineer to interpret and adapt these basic principles to each structure using experience and good judgment.

Because of the great variability and complexity of individual structures, it is impractical and beyond the scope of these Requirements to go into the detail necessary to specifically cover all of the possible variations in structure response. The dynamic characteristics of the structure, behavior of structural elements, the variability in ground motion, and the types of soil all affect building response. Thus, both latitude and responsibility for the exercise of judgment must be given to the engineer.

Many types of structures require seismic design procedures that may differ, possibly substantially, from these Requirements. These types include, but are not limited to, bridges, dams, retaining walls, docks, industrial and refinery structures, and offshore platforms.

Section 109, Nonbuilding Structures, provides guidance for the design of a limited class of these nonbuilding structures. Section 109 should be used in conjunction with existing regulations, and/or standard criteria developed for such structures. When a particular structure is not within the group treated under Section 109, the engineer must establish criteria appropriate for the particular structure and requirements for performance and reliability.

C101.1.4 ***Regulations That Exceed These Requirements.*** Some regulations have design requirements that exceed those set forth in these Requirements. For example, Title 24 of the California Code of Regulations gives requirements for providing enhanced life safety and property protection for schools and hospitals. Additionally, hospitals are expected to remain operational after an earthquake. However, because of the real variability in seismic response and the lack of a consensus on criteria, most general codes do not specifically give provisions for damage control since they may not be sufficiently reliable.

C101.1.5 ***Damage Control.*** Where damage control is desired, the design must not only provide sufficient strength to resist the specified seismic forces, but also provide the proper stiffness or rigidity. Control of damage is very complex. Some structural elements are sensitive to acceleration making floor acceleration the key design consideration, while others are sensitive to interstory deformations making drift the key design consideration.

Damage to nonstructural elements may be minimized by proper limitation of deformations, by careful attention to detail, and by providing proper clearances for exterior cladding, glazing, partitions, and wall panels. Some nonstructural elements can be separated or floated free and allowed to move independently of the structure. If they are tied rigidly to the structure, these elements need to be capable of accommodating the deformations or accelerations imparted by the elements' base motion; otherwise, damage must be expected. It must be recognized that major earthquake ground motion can cause interstory deformations several times larger than those calculated with the seismic design forces given in these Requirements. If a performance-based design method, such as damage control, is desired, the engineer is cautioned that project specific design criteria to achieve the desired performance, e.g. control damage, must be developed as a supplement to these Requirements.

C101.2 Wind and Seismic Forces

Wind and seismic forces are fundamentally different. Wind force results from aerodynamic pressures applied to an exterior surface of a structure, whereas seismic forces result from the inertial response of a structure to the accelerations and displacements from earthquake ground shaking. Wind pressures and the resulting forces tend to have somewhat predictable upper limits; the upper limits of seismic inertial forces, on the other hand, tend to be unpredictable. Furthermore, the seismic design forces set forth in these Requirements are based on inelastic behavior in the structure; the forces resulting from the inelastic behavior are much lower than the seismic forces that would occur if the structure remained elastic.

Seismic ground shaking can cause several reversing cycles of inelastic deformation. Thus, even when wind pressures govern the stress (strength) or drift design, the structural lateral force resisting system must conform to the special detailing requirements for seismic systems in order to accommodate the inertial forces and displacements from an earthquake. These detailing requirements are intended to allow the lateral force resisting system to deform inelastically without catastrophic failure and to provide the necessary inelastic energy absorption and toughness necessary to resist actual seismic forces. Actual seismic forces are expected to exceed the specified seismic design forces.

C101.3 Continuous Load Path

While providing a continuous load path is an obvious design requirement for acceptable structural performance, experience shows that this fundamental requirement is often overlooked, leading to significant damage and collapse. Because it is one of the most fundamental considerations in seismic design, this requirement is stated at the beginning of these Requirements to emphasize its importance. Detailed Requirements for selecting, or identifying and designing, this load path are given in the relevant design and material sections.

C101.4 Basis for Seismic Design

This Requirement for clearly stating the basis for the structure's seismic design has two main objectives:

1. Ensure that the engineer has fully identified the particular lateral force resisting system and its appropriate design levels.
2. Allow the design basis to be fully identified for the purpose of future modifications and changes in the structure.

C101.5 Computer Calculations

This Requirement is intended to be used where extensive computer outputs are obtained from analysis routines. When one- or two-page computer outputs only are included directly in the calculations, these Requirements may be modified to fit the circumstances. Documentation must be sufficient for a third party to understand the basis for, and sequence of, the analysis.

C101.5.1 The intent, where computerized calculations are included in the design data, is to require sufficient computer documentation so that the assumptions made can be understood and so that it is possible to recreate the results, if necessary.

C101.5.2 Documentation must be sufficient for another person (or third-party reviewer) to employ the specified computer program and understand the assumptions made by the program's developers. Care must be given to identify the version of the software used, along with a statement that the submitter believes the program is functioning properly. When standard software is modified, these changes must be documented and discussed.

C101.5.3 Both the input and the computer output must be provided, making clear what quantities were generated by the program.

C101.5.4 This Requirement that the computer output be signed by the engineer of record is to ensure that the engineer confirms the appropriateness of the input data for the building under consideration and gives the results sufficient review.

C102 UBC §1625 Definitions

Base. Previous editions of the SEAOC Blue Book permitted the first complete floor at grade to be considered as the base for the structure. The reasoning behind this was quite direct — most buildings have the first floor at grade. For buildings having basements, even though grade was considerably below the first floor, the rigid nature of the basement structure produced a "dynamic setback" at the first floor for the longer period structure above. The basement story had very little effect on the period of vibration of the total building.

The first floor base was used for the purpose of computing a building period T from which the C coefficient and ultimately the base shear V were computed. Little guidance was given on the application of this shear load as applied to the base, but most engineers provided for a path of sufficient strength to transfer this base shear between the structure and the soil.

The first floor is still the base for the majority of buildings. However, it is now mandatory to consider the mechanism by which earthquake forces are transmitted from the soil into the building. The interface at which this is accomplished is now considered the base. Depending on the type of building under discussion, it may not be unusual to consider a rigid basement floor slab or the bottoms of building footings as the base.

The selection of a base for a building on a sloping site presents special problems. Usually, very rigid structural elements should be utilized below the highest contact with the site to equalize the stiffness distribution in the building plan, thereby minimizing torsion.

Bearing Wall System. This class of structural systems includes those that were known as the box system in previous versions of the SEAOC Blue Book. See the details in Section 104.6.1.

Boundary Element. The function of this element is to resist significant tension and compression stresses in shear walls and diaphragms.

Collector. Also referred to as a *drag member* or *drag strut*.

Design Level Earthquake Ground Motion. When this term is used in these Requirements, the following definitions may be used to represent the elastic response effects:

1. ZC= multi-mode response envelope spectrum for earthquake motion with a 10 percent probability of exceedance in 50 years for the static lateral force procedure (Section 105.2). The value of ZC times the gravity constant g

provides acceleration response. The C value should be evaluated with the appropriate soil site coefficient from Table 104-2 (see Requirements).

2. Z times the ordinate value of the normalized spectral curve given in Figure 104-2 (in the Requirements) for the appropriate soil site conditions. Z is used in the dynamic lateral force procedure (Section 106). The value of Z times the ordinate of the normalized spectral curve times the gravity constant g provides the acceleration response spectrum value.

   or:

3. A site-specific acceleration response spectrum with 5 percent damping for earthquake motion with a 10 percent probability of exceedance in 50 years for use in the dynamic lateral force procedure (Section 106).

Horizontal Bracing System. A horizontal steel bracing system acting primarily as a horizontal diaphragm should be designed for the prescribed seismic forces based on allowable stresses. The design of horizontal bracing need not conform to requirements under UBC §2211.8, except for the transfer of seismic forces from in-plane discontinuous lateral force resisting elements.

Maximum Capable Earthquake. Within the presently known tectonic framework, the maximum earthquake that appears capable of affecting the structure.

Special Moment Resisting Frame. A lateral force resisting system made up of designated moment resisting frames specially detailed to provide ductile behavior, and that comply with the Requirements given in Chapter 4, Reinforced Concrete, or Chapter 7, Structural Steel. While not all frame elements of the space frame are required to be part of the lateral force resisting system, the designated frames should be sufficiently numerous and extensive so as to provide adequate redundancy.

Strength. Also known as the *capacity* or the *capability to resist loads* when evaluated using the "strength design" basis for reinforced concrete or the equivalent of this basis for other materials.

Weak Story. Given the appropriate design basis and the "working stress capacity" or "strength" for the lateral load resisting elements of each story, the comparison of the relative capacities or strengths of stories should consider the individual load-deformation behavior of the elements and of the diaphragms. For the assumption of rigid diaphragms, the story strength may be taken as the sum of the individual strengths of resisting elements for the story. This sum should be at a consistent deformation level so that it can be verified that all the elements are able to provide their resistance contribution at the common deformation value. If, for example, some elements are strong but substantially less rigid than the other elements, then the full strength of these less rigid elements may not contribute to the story strength. For flexible diaphragms, it may also be necessary to consider the relative interstory strength of those important resisting elements that support significant amounts of tributary diaphragm area.

Whatever method is used to define the weak story condition, the primary purpose should be to detect if story inelastic demand due to the response effects of the maximum expected level of ground motion would be excessive. Therefore, the governing criterion for strength comparison should be between the story shear required to fully yield or create a mechanism in the story being evaluated and the total lateral load required to create a mechanism in the structural system above that story (see additional discussion on weak stories in Sections 104.5.2 and 104.11.4).

C103 UBC §1626 Symbols and Notations

No Commentary provided.

C104 UBC §1627 Criteria Selection

This section of the Requirements is intended to provide guidance to engineers as they make key decisions regarding lateral force requirements, analysis, and related parameters. Such guidance is especially important at the initial stages of design, where many options may be under consideration.

C104.1 Basis for Design

This paragraph is an introduction to the whole Section 104, indicating the principal topics covered in criteria selection considerations.

C104.2 Seismic Zones

Seismic Zones defined in Figure 104-1 are based on effective peak ground accelerations (EPA) projected for the region in question. In the previous edition of these Requirements, the Z value was a normalized coefficient with a maximum value of 1.0 for Zone 4. The Z factor now corresponds numerically to the EPA of the Zone in question. This was done to allow these Requirements to be applied where different probabilities of exceedance in a given time period are appropriate, or applied to geographic areas outside California where EPAs are available (see Commentary on Seismic Zone Factor Z in Section 105.2.1). It should be noted that, in other parts of the country, differences in sources, attenuation properties, and site response characteristics, particularly as represented by the effects of the effective peak velocity (EPV), may require a change in the spectrum shape (see Commentary Section C106.2). The terms EPA and EPV were originally defined in the ATC 3-06 Commentary [ATC-3-06, 1987], and a related discussion is given here.

Based on the representative behavior of strong ground motion records, the shape of response spectra can be described by a constant spectral acceleration portion (S_a) and a constant spectral velocity portion (S_v), as shown on the tri-partite logarithm plot, Figure C104-1 (taken from the ATC 3-06 document). For the data and geotechnical knowledge existing at the time of the writing of ATC 3-06, the following relations were used to convert these constant spectral levels to ground motion parameters: effective peak ground acceleration (EPA)$=A_a$; expected peak ground velocity (EPV) and the velocity related acceleration (A_v). Given the forecast values of the 5 percent damped response spectrum values S_a in inches per second squared, and S_v in inches per second, corresponding to a hazard level of 10 percent in 50 years, $A_a = S_a/2.5g$ in g's, where 2.5 is the spectral amplification factor for the 5 percent damped spectrum, and g is 386 inches per second squared, $\text{EPV} = S_v/2.5$, and $A_v = \text{EPV}/30$ in g's.

The constants in these relationships and the methods of assessing the spectral levels of S_a and S_v are subject to change as more records and knowledge are accumulated. The intent is to provide the best estimate of future response spectrum levels at a specified hazard or probability of exceedance level. In order to attain this objective, it is necessary to consider:

1. The possible differences between the peaks of the accelerogram and the actual ground motion delivered to the overall footprint and geometry of the building foundation.
2. How the actual building system may differ from its linear elastic analytical model and the associated single-degree-of-freedom system used to obtain the response spectrum.
3. The fact that statistical averages of past earthquake records, along with judgmental modifications, are used to forecast the peak ground accelerations, velocities, and response spectrum shapes.

These listed conditions must enter into the process of assigning the A_a and A_v values because the peak ground acceleration (PGA) is not a consistent scaling factor for the spectral values of S_a and S_v. For example, a low magnitude, short duration record measured close to the earthquake epicenter may have a low ratio of S_a to PGA, while a large magnitude, long duration event measured far from the source would show a low PGA value with proportionally large S_a and even larger S_v values.

C104.3 Site Geology and Soil Characteristics

Local geology and soil characteristics influence the ground motions at a site. The significance of this influence varies from essentially nil to very large. A graphic example is provided in the Mexico City earthquake of September 1985: the lake bed amplified motions were substantially larger compared to adjacent rock sites [ATC 3-06, 1987; U.S. Army, 1992; U.S. Army, 1986; U.S. Navy, 1982; Zsutty and Shah, 1981; Bertero, 1986]. See Commentary Section 104.8.2, Item 4 for further discussion.

Past editions of these Requirements provided formulas that quantified the site factor, S, in terms of the building period and a representation of the fundamental period of the site. An alternate procedure was suggested by the ATC 3-06 study and is adapted herein. The quantification given by the Formulas 1-4 and 1-4A of the 1980 edition of these Requirements have been subsequently evaluated as providing a false sense of understanding not consistent with current information; therefore, their use was not continued (see Section C105.2.1 for further Commentary on the site coefficient S).

C104.4 Occupancy Categories

In previous editions of these Requirements, special occupancy consideration was given for only those essential facilities now considered as Occupancy Category I: the *I*-factor increased lateral design forces by 50 percent.

Two additional categories are now included: hazardous facilities (Category II) and special occupancy structures (Category III). These categories are defined in Table 104-3 (see Requirements). All three categories require special review, inspection, and construction observation (Requirements are provided in Chapter 2). Category I and II facilities have 25 percent increased lateral force requirements, through an I factor of 1.25 and an $I_p = 1.5$, while Category III facilities have $I = 1.0$ and $I_p = 1.0$.

It was judged that an increased force factor of 25 percent for structures in Categories I and II, coupled with special design and construction review/inspection/observation, should provide the same level of overall seismic performance that was intended by the former 50 percent increase in load. In the judgment of SEAOC, the details of design and construction often dominate seismic performance. Therefore, increasing these aspects for essential facilities can improve performance more effectively than relying solely on increased design force levels to achieve this end. The higher I_p used for elements, equipment, and components reflects lower redundancy, damping, and ductility in these items relative to the structure.

When describing Category I, II, or III facilities, recall that the Requirements are intended as minimum requirements only. Additional levels of protection may be obtained by providing additional energy dissipation capacity, redundancy in lateral force resisting systems, special detailing for damage control, construction quality assurance, and by increasing design force levels (see Commentary Section C105.2.1 on Importance Factor I).

C104.5 Configuration Requirements

Irregularities in load paths and in structural configuration are major contributors to structural damage and failure due to strong earthquake ground motion [Stratta, 1987].

Commentaries to previous editions of these Requirements and to ATC 3-06 have described undesirable configuration characteristics that are to be avoided wherever possible. Previous Requirements have required dynamic analysis of structures that have significant irregularities. However, neither the approach to dynamic analysis, nor the level of irregularity that requires such analysis was specified.

Extensive engineering experience and judgment are required to quantify irregularities and provide guidance for special analysis. As yet, there is no complete prescription for all special analysis considerations for irregularities. However, guidelines for quantifying irregularities and for special analysis are outlined in Sections C104.5.1 and C104.5.2 below. These guidelines are especially beneficial considering that buildings are becoming increasingly complex, and their design requires additional care to achieve the seismic performance called for in Commentary Section C101.1.

These guidelines reflect the increase in engineering knowledge and the understanding of building response and the effect of irregularities. Much of the increase in knowledge has come through the increasingly common use of computer analysis.

In reviewing the requirements for dealing with irregularities, it was recognized that the use of the elastic model in dynamic analysis can detect and correct the response effects of only certain types of irregularities. The vertical irregularities covered in Table 104-4 (see Requirements), types A, B, and C, are cases in point. Other irregularity effects are mitigated through design details and other analytical considerations such as limitations on story drift ratio differences and torsional effects (Table s 104-4 and 104-5 in Requirements).

C104.5.1 ***Regular Structures.*** Regular structures, designed to the prescribed seismic design load levels, are expected to meet the objectives of Commentary Section C101.1. Two important concepts or assumptions apply to regular structures.

1. The linearly varying lateral force distribution given by Equations 105-6, 105-7, and 105-8 is a reasonable and conservative representation of the actual dynamic response inertia force distribution due to earthquake ground motions. Given a common base shear value, the equivalent static force distribution provides a reasonable envelope of the forces and deformations due to the actual dynamic response.
2. When design of the elements in the lateral force resisting system is governed by the specified seismic load combinations, then the cyclic inelastic deformation demands during a major level of earthquake ground motion will be reasonably uniform in all elements, without large concentrations in any particular part of the system. It is assumed that the inelastic deformation demands are proportional to and reasonably approximated by the deformations of a fully elastic (nonyielding) model of the structure when subject to major levels of ground motion. The acceptable level of inelastic deformation demand for a given material and system is, therefore, represented by the R_w value for the system, and the seismic design level at a working stress basis is given by the fully elastic response divided by the R_w value, as given by the base shear Equation 105-1.

   When a structure has irregularities, then these concepts, assumptions, and approximations may not be reasonable or valid, and corrective design factors and procedures are necessary to meet the design objectives.

The reduction of column size, wall size, and reinforcing and/or bracing, as the required story shear resistance decreases with the height level of the story, can sometimes result in an abrupt change in story strength, and a large concentration of inelastic behavior can occur at the level of element reduction. It is recommended that these reductions be staggered or distributed over different levels so as to avoid this inelastic concentration that holds potential for story failure.

C104.5.2 ***Irregular Structures.*** Vertical and plan irregularities can result in loads and deformations significantly different from those assumed by equivalent static procedures; those distributions of mass, stiffness, or strength that result in earthquake forces and/or deformations over the height of the structure, which are significantly different from linearly varying distributions, are designated as vertical irregularities. Those plan or diaphragm characteristics that result in significant amounts of torsional response, diaphragm deformations, or diaphragm stress concentrations are designated as plan irregularities.

The lack of a direct path for force transfer is referred to as an *irregularity in force transfer*, and an example would be where a lateral force resisting element is offset or discontinuous. This condition can cause a concentration of inelastic demand, and can occur even when no irregularities in plan or elevation exist.

It is most important for the engineer to understand that irregularities create great uncertainties in the ability of the structure to meet the design objectives of Commentary Section C101.1. Therefore, the presence of requirements to accommodate irregularities shall not be construed as an endorsement of irregular structures, a "catalog" of imaginative structural forms, or a manual for their design. These Requirements are intended only to make designers aware of the existence and potential detrimental effects of irregularities, and to provide minimum requirements for their accommodation. If an irregularity cannot be avoided or eliminated by design changes, then the engineer must not only apply the provisions given in these Requirements, but also consider the ability of the structural system to provide the objective of life safety protection at a major level of earthquake ground motion; this is particularly necessary for cases of force transfer path irregularity. While incorporating requirements for irregularity runs the risk of their misuse as a "design manual" for these undesirable structural features, they do serve to correct a far more serious danger. Designers must be aware of the existence and effects of irregularities. They must not employ conventional methods, as if the building were regular, without accommodating dynamic response and/or force transfer deficiencies induced by these irregularities.

The various ratios and related criteria stated to define irregularity in Table s 104-4 and 104-5 (see Requirements) have been assigned by judgment based on the interpretation of past earthquake damage effects and design experience. In many cases there is no way, short of an inelastic dynamic analysis of the complete structural and nonstructural system, to assess or quantify the effects of these irregularities. It is also essential that sound professional judgment be used to formulate these analyses and to interpret their results. However, since such detailed types of analysis are generally not economically justifiable, it is the intent of these guidelines to assist the engineer in detecting particular problem areas and correcting their detrimental effects. For this reason the guidelines given for the assessment of irregularities are qualitative in nature and must be interpreted with sound engineering judgment. Further, it is of foremost importance in the design process to eliminate, or at least minimize, significant irregularities.

Extended commentary on the description and response effects of irregularities is given in Commentary Section C104.11.

C104.6 Structural Systems

This section defines the general types of structural systems that may be used to resist earthquake forces. These systems are further classified in Table 104-6 (see Requirements), and maximum values of R_w are specified for each classification. The R_w values have been assigned according to the relative capabilities of building systems to provide energy dissipation in the inelastic range (see structural system factor R_w in Commentary Section C105.2.1 and in Appendix 1C to Commentary Chapter 1). R_w values for structures other than buildings are given in Table 104-8 in Requirements.

In any structural system, it is preferable that the individual lateral force resisting elements be located on separate vertical planes for each direction of loading. It is important that adequate resistance to torsion be provided both by the strength and arrangement of these elements in the plan of the structure.

The particular requirements necessary to qualify for a given structural system type and its corresponding R_w value follow.

C104.6.1 ***Bearing Wall System.*** This type of structural system is characterized by shear wall or braced frame lateral force resisting elements that carry substantial vertical loads which, if they fail, eliminate the vertical load capacity of a substantial portion of the structure and/or lead to vertical instability. The presence of minor load bearing walls in a building that would normally be classified as a Building Frame System (Section C104.6.2) does not necessarily mean that the building should be categorized as a Bearing Wall System. The reasoning is that the action of a multi-storied building is not significantly influenced by the presence of minor bearing walls around a stairwell, for example. Also, in a tall building with setbacks, the completeness of the frame for the tower, when carried through to the foundation, is not adversely affected by bearing walls in the base structure below the tower.

Note that an $R_w = 8$ is given for buildings that are of repetitive stud and joist frame construction, do not exceed three stories in height, and use horizontal diaphragms and plywood vertical shear panels to resist lateral forces. These buildings have demonstrated excellent resistance to strong ground shaking when designed, detailed, and constructed to good engineering standards. To qualify, the diaphragms and shear panels for this type of building must be made of a combination of joists, studs, and plywood sheathing materials with an established record of acceptable performance as shear elements. Walls and diaphragms with concentric diagonal bracing do not qualify for this $R_w = 8$ value. Also, $R_w = 8$ is based on the condition that the diaphragm does not have excessive openings and/or discontinuities and the wall system is reasonably continuous and complete down to the foundation level. Structures with large openings, such as lower level garage spaces having only columns and narrow wall piers supporting the upper stories, should use the $R_w = 6$ or a lower value.

The prescribed limitations for the height-to-width ratio for shear panels should be applied for the total height of the panel above the level where width is determined, and not applied for only the story height.

In Table 104-6 (see Requirements), the system A.4.a, "Braced frames where bracing carries gravity loads—steel," is distinctly different from B.4.a, "Concentric braced frames—steel." In system A.4.a, the bracing elements perform the dual function of resisting both gravity loads and lateral seismic loads. If inelastic behavior and energy dissipation occur in the bracing elements, then their ability to carry tributary gravity loads may be lost. In the B.4.a system, the gravity load is carried by the columns, and the vertical load capacity of the system is not endangered by energy dissipation in the braces. Refer to UBC Sections 2211.5.1, 2211.9, and 2211.10 for Requirements intended to maintain the load-carrying capacity of columns after bracing elements have yielded.

Chevron bracing was an example of the A.4.a system in previous editions of these Requirements. In the current Requirements, chevron bracing is a B.4.a system. If the beam hangs from braces above (e.g., an inverted chevron), energy dissipation in the braces does not endanger the gravity load capacity of the braces; if the braces support the beam from below (e.g., an inverted *V*), the beam must be capable of carrying the tributary gravity loads without the support of the braces.

C104.6.2 ***Building Frame System.*** This system consists of a vertical load-carrying space frame with lateral load resistance provided by braced frames or nonbearing shear walls. This definition does not require that absolutely all the vertical load be carried by the vertical load-carrying frame (see Commentary Section C104.6.1, above, relative to the presence of minor load-bearing walls).

While there is no requirement to provide lateral resistance in the vertical load framing, it is strongly recommended that nominal moment resistance be incorporated into the vertical load frame design. In structural steel, this might be in the form of nominal moment resisting beam flange and/or web connections to the columns. In reinforced concrete, the nominal moment resistance inherent in cast-in-place concrete may be considered sufficient to qualify for this system, while most types of precast concrete systems would not. In reinforced concrete, continuity and full development and anchorage of longitudinal steel, along with stirrups over the full length of beams would be good design practice. (Refer to ACI 318-89, 1992 Rev., Chapter 21, for specific requirements for frames not designated as a part of the lateral force resisting system.) The vertical load frame provides a nominal secondary line of defense, even though all specified lateral forces are resisted by other seismic resisting structural systems. In addition, the deformation compatibility Requirements of Section 108.2.4 must be met. The presence of a frame can provide vertical stability to the building and prevent collapse after damage to the shear walls or braced frames. The frame also acts to tie the building together and redistribute the lateral force to undamaged elements of the lateral force resisting system.

C104.6.3 ***Moment Resisting Frame System.*** As in the case of the building frame system, it is not necessary that all the vertical load be carried by the vertical load-carrying frame. The entire lateral force stipulated must be capable of being resisted by moment resisting frames. To qualify for $R_w = 12$, the frame shall be a special moment resisting frame of structural steel conforming to Chapter 4, Reinforced Concrete, or of specially detailed reinforced concrete conforming to Chapter 7, Structural Steel.

C104.6.4 ***Dual System.*** A structural system with the following features:

C104.6.4.1 The vertical load-carrying space frame shall be substantially complete. Special details are required according to the R_w value and seismic zone.

C104.6.4.2 and C104.6.4.3 The shear walls or braced frames and moment resisting frames of the lateral force resisting system shall conform to both of the following criteria:

1. The frame and shear walls or braced frames shall resist the total required lateral force in accordance with relative rigidities considering the interaction of the walls and frames as a single system. This analysis shall be made in accordance with the principles of structural mechanics that consider the relative rigidities of the elements and torsion effects in the system. Deformations imposed on members of the frame by the interaction with the shear wall or braced frame shall be considered in this analysis. In previous editions of these Requirements, shear walls or braced frames were required to be designed for 100 percent of the required lateral force. This was a simple rule that permitted design without the necessity of evaluating the interaction effects with the moment resisting frame. With the availability of computer programs for the analysis of the complete structure model, the walls or braced frames can be designed for more realistic interaction forces, and the 100 percent rule is no longer needed. While this may result in a somewhat reduced loading when compared to previous Requirements, there are extra design Requirements set forth in Section 104.6.4 to ensure attainment of the performance objectives for the structure.
2. The special moment resisting frame acting independently shall be designed to resist not less than 25 percent of the total required lateral force, including torsion effects. Columns of the frame system may also function as the boundary elements of shear walls. As such, these columns must be designed to resist the vertical forces resulting from overturning moment in the shear wall along with the load effects associated with the frame system.

C104.6.5 ***Cantilevered Column System.*** A cantilevered column system consists of an assemblage of cantilever column elements. The cantilevered column system is essentially an inverted pendulum. The R_w of 3 was selected to be compatible with the inverted pendulum system allowed for nonbuilding structures.

The inverted pendulum-type system listed in Table 104-8 (see Requirements) is not limited to nonbuilding structures, as implied by the table. In fact, this type of structural system is common for multi-family residential occupancies over carports, strip shopping center storefronts, and single family dwellings on ocean or hillside lots. Inclusion of this class of structures does not imply that this is a preferred type of system, but rather recognizes the existence of the cantilever column system as a separate category with unique characteristics.

Unlike some of the other seismic force resisting systems, cantilevered columns have very limited redundancy. Typically, this system does not have redundant elements and multiple load paths, nor does the system have added damping, strength, or rotational stiffness developed by the nonstructural elements.

In cantilever column systems, the column elements acting in cantilever action often resist all lateral forces as well as provide support for the gravity (vertical) loads. Hence, there is no independent vertical load-carrying system and the failure of the primary lateral system compromises the ability of the structure to carry gravity loads.

Overstrength in cantilevered column systems is minimal because the ability to form a progression of plastic hinges is limited. Hence, design for higher strength and stiffness (by use of a low R_w) is necessary to reduce the high ductility demands imposed on the columns.

This type of system should be used only when the use of other more desirable systems is not feasible. If this type of system must be used, care should be used to ensure that deformation compatibility is checked at ultimate seismic induced displacements for both structural and nonstructural elements. Deformation compatibility checks are particularly important if the cantilever column system is used as part of a mixed system, e.g., cantilever columns with plaster or gypsum board shear walls.

C104.6.6 ***Undefined Structural System.*** In the previous Requirements, all other systems were assigned a $K=1.00$ value (equivalent to $R_w=8$). Undefined structural systems are those not listed in Table 104-6 (see Requirements) and must have their design basis justified in accordance with Section 104.9.2.

C104.6.7 ***Nonbuilding Structural System.*** In the previous editions of these Requirements, structures other than buildings were all assigned a $K=2.00$ value (equivalent to $R_w=4$). Section 109 and Table 104-8 (see Requirements) now prescribe design procedures and bases for various types of nonbuilding structures.

C104.6.8 ***Comments Related to Systems in Section 104.6 and Table 104-6.*** Requirements for Seismic Zones 1 and 2: Table 104-6 (see Requirements) and its system Requirements now apply to all Zones 1 through 4. Previous Requirements applied only to Zones 3 and 4.

In Zones 1 and 2, the expected ground motion having the same probability of exceedance as in Zones 3 and 4 will not produce the same large inelastic response demands that would occur in Zones 3 and 4. When the seismic force is small compared to the gravity load and wind load requirements, there is more reserve strength to resist seismic forces. Therefore, in Zones 1 and 2, it is permitted to use systems and configurations with lower energy dissipation capabilities, with appropriately reduced R_w values—such as concrete intermediate and ordinary moment resisting frames, and concrete braced frames. It must be recognized, however, that major earthquakes with very low probability of occurrence can occur in some areas of Zones 1 and 2. The level of ground motion from the largest capable event in these Zones may be of the order of two or more times the level of the expected ground motion having 10 percent risk of exceedance in 50 years. Therefore, the decision to reduce special detailing requirements and permit the use of systems with low inherent energy dissipation capabilities entails a risk of very serious damage or collapse if the very rare, but large, event were to occur. This may be economically feasible in those instances involving only a small amount of life hazard; otherwise, the risk may not be acceptable. In areas of high seismicity, seismic Zones 3 and 4, this problem does not exist for two reasons:

1. Only systems with high energy dissipation capabilities are allowed.
2. The largest capable event (with, say, a 25,000-year return period) may produce ground motion levels that are approximately 1.5 times higher than the level considered in preparing these Requirements.

Concrete braced frames are restricted to lower seismic zones: There is a lack of experience with this system, and little research exists on which to base detailing requirements for higher zones.

C104.6.9 ***Damage Control Considerations.*** Damage to, and repairability of, the structure was not a direct consideration in development of the R_w factors. Thus, it may be appropriate for the engineer to consider damage control and increase the level of loading, stiffness, and detailing requirements if damage control and/or decreasing the cost of repairing the damage is a consideration.

C104.6.10 ***Need for Redundancy.*** It is *strongly* recommended that the lateral force resisting system be made as redundant as possible; multiple lines of resistance are preferable to perimeter resistance only, and multiple bents or bays of resistance in each bracing line are much more desirable than a single bent or bay. Good torsional rigidity is also essential. The objectives are to:

1. Create a system that will have its inelastic behavior distributed nearly uniformly throughout the plan and elevation of the system.
2. Have such a degree of redundancy that softening or failure of a particular element will result in redistribution of the load to the associated redundant elements, without the possibility of collapse.

C104.6.11 ***Space Frames.*** Several aspects of the space frame and the moment resisting or special moment resisting frames require discussion. It is essential that the vertical load-carrying space frame be substantially complete to qualify the structure for an R_w value greater than 6. The engineer should have the latitude to designate those parts as the moment resisting or special moment resisting frame, as long as it satisfies the design requirements and provides the intended behavior of the selected type of lateral force resisting system. The purpose is to allow the engineer to select the most effective configuration for the lateral force resisting system. However, the need for redundancy should be a primary consideration in this selection process.

For those portions of the space frame that are not part of the designated lateral force resisting system, the deformation compatibility Requirements of Section 108.2.4 must be complied with. Also, if the materials, type, or configuration of these portions of the space frame are such that brittle failure and/or collapse could occur if the inelastic displacements were to exceed the factored values prescribed by Section 108.2.4, then extra reinforcing details should be provided to prevent brittle failure, or the amount of inelastic displacement must be controlled by extra stiffness and strength in the designated lateral force resisting system. See related Commentary Sections C108.2.4 and C408.

C104.7 Height Limits

Building codes have long prescribed height limits, principally for purposes of fire protection. A single 160-foot level height limit was imposed on many systems for seismic protection by previous editions of these Requirements. In this edition, the limits of some systems were reduced and others were raised. In general, special moment resisting frame systems have no height limits as they have had in the past. Dual systems have either a 160-foot limit, or no limit, depending on the combination considered.

Many systems where lower limits have been prescribed are of materials where the practice of the industry has been, in general, to limit height approximately to the values provided. A 50 percent increase in height limit is allowed under circumstances where the Requirements of Section 104.9.2 and the special review requirements for design and construction are imposed. This illustrates how an increase in knowledge of expected performance and quality assurance can result in a relaxation of previous established limits.

C104.8 Selection of Lateral Force Procedure

The philosophy underlying this section is that dynamic analysis is always acceptable for design. The static lateral force procedure (Section 105) is allowed only under certain conditions of regularity, occupancy, and height. For example, regular buildings under 240 feet in height may be designed using the static lateral force procedure. The following considerations apply for cases where it is not immediately obvious whether the structure requires dynamic analysis or may use the static lateral force procedure.

Assessment of Irregularity. It must first be determined whether the system qualifies for the vertical irregularity exception in Section 104.5.2.1, and/or whether the displacements due to torsional effects are below the plan irregularity limits of Type A in Table 104-5 (see Requirements). Given the preliminary trial configuration and member sizes for the structural system, the appropriate equivalent static force levels should be applied to the structure, along with the corresponding vertical load combinations. This analysis provides the displacement or drift information to make this determination.

If at all possible, the design and/or configuration should be modified to conform to the displacement limits required for regularity, and, of course, any of the other irregularity features of Table s 104-4 and 104-5 (see Requirements) should be eliminated if possible. Otherwise, the design is subject to the following penalties and restrictions:

1. Vertical irregularity Type E in Table 104-4, known as a weak story, represents the existence of any story whose lateral resistance is less than 80 percent of the story above. For structures whose height exceeds 30 feet, stories having lateral resistance less than 65 percent of the story above are prohibited (Section 104.9.1).
2. Discontinuous lateral force resisting systems, such as discontinuous shear walls, require special design of supporting columns (Section 105.7.2).
3. The higher mode overturning effect represented by the quantity F_t may be neglected at the soil interface for regular structures. It may not be neglected for irregular structures (Section 108.2.10).
4. Plan irregularity Type A in Table 104-5 represents a torsional irregularity that occurs when the center of mass has a large eccentricity relative to the lateral force resisting elements. This irregularity carries a penalty of increased accidental torsion, in which an amplified accidental eccentricity must be used in design (Equation 105-9).
5. More stringent stress limits are required for connections of diaphragms to vertical elements and to drag members when plan irregularities exist (Section 108.2.9.7).
6. Concurrent application of earthquake forces in two orthogonal directions is required when torsional irregularity Type A of Table 104-5 exists in both principal horizontal directions. There is a similar Requirement for application of earthquake forces in two orthogonal directions when the structure has plan irregularity Type E in Table 104-5. This latter type of irregularity exists when the vertical elements are neither parallel to, nor symmetric about, a set of principal horizontal orthogonal axes (Section 108.1.3).
7. The prescribed base shear may be reduced by 10 percent for regular structures when a dynamic analysis is performed. This reduction is not permitted for structures with irregular features (Section 106.4.1.3).

C104.8.1 ***General.*** In these Requirements, the results of a dynamic analysis are normalized or scaled (Section 106.4.1) to the levels required by the static force procedure of Section 105. Therefore, the basic information provided by dynamic analysis consists of the particular distributions of forces and deformations as they might differ from those provided by the static load distribution of Section 105.

For the structures described in this section, the static lateral force procedure was judged to provide an adequate envelope for the pattern of load distribution and the corresponding deformations. Note that all irregular wood structures are included in the height limited group of Section 104.8.2, Item 3. Wood structures have, in general, shown good performance in earthquakes due to their inherent energy dissipation characteristics and the usual presence of redundancies. However, the engineer should still recognize the significance of the effects of irregularity and take appropriate measures to ensure performance consistent with the objectives of this document.

C104.8.2 *Static Procedure*

Item 1. While these Requirements exempt short buildings from the dynamic analysis requirements, the engineer should be aware of the need to evaluate the dynamic characteristics of buildings not more than 65 feet in height where properties and/or characteristics may create dynamic force and deformation patterns that differ significantly from those resulting from the load distribution of Section 105.

Item 2. Refer to Section 105.3.1, Item 2 for a design procedure that may be used for structures meeting the limitations of this section. Here, structures having the same R_w value for both the platform and tower portions may be considered as a special case of a mixed R_w value system.

C104.8.3 *Dynamic Procedure*

Item 1. Dynamic analysis is required for tall structures over 240 feet in height in order to determine the possible effects of higher mode response on the force distribution and deformations.

Items 2 and 3. Dynamic analysis is required because the cited irregularities invalidate the assumptions applicable for equivalent static force analysis.

Item 4. In September 1985, Mexico City was shaken by a great earthquake centered some 400 kilometers to the southwest. The shaking experienced in Mexico City during this event varied markedly with subsurface soil conditions. This shaking was most intense within a region underlain by an ancient lakebed comprised of soft clay deposits. The recorded ground motions within this lake bed region had peak accelerations that were only moderately strong (0.17g); however, these motions lasted nearly 2 minutes and exhibited nearly harmonic behavior with a period of about 2 seconds. As a result, the spectral amplitudes in this period range for motions recorded on the lake bed were much larger (by factors of 15 to 20) than the corresponding spectral amplitudes for motions recorded on rock sites at comparable epicentral distances [Bertero, 1986]. Because of this, the most significant damage was confined to the lakebed region and occurred in buildings that were 5 to 15 stories in height with small-strain natural periods of about 0.8-1.0 seconds. When these buildings began to crack and yield during the shaking, their natural period began to lengthen. As this lengthening period approached the dominant period of the lakebed motions (approximately 2 seconds), the structural response intensified as it entered into resonance with these ground motions. The long duration of the nearly harmonic ground motions provided ample time for development of resonance conditions, resulting in many cycles of intense shaking of the buildings. This, in turn, led to a progressive increase in building damage and to eventual collapse in many cases.

It is important to recognize that the volcanic origin, mineralogy, and extremely high water content (100-500 percent) of the lakebed clay deposits are unique to the Mexico City area. These factors have contributed to the essentially linear and lightly damped nature of the lakebed's earthquake motions and to their extremely large amplification characteristics. In addition, the location of the lakebed soil deposits in a basin of rock-like material contributed to the usually high long-period amplifications that were observed [Singh, 1988; Dobry and Vucetic, 1987].

The geologic characteristics of the Mexico City lakebed deposits are not typical of most Type S_4 sites in California. Therefore, it is unlikely that Type S_4 California sites would exhibit ground response characteristics comparable to those observed in Mexico City. Nevertheless, because of the limited understanding of the behavior of soft clay (Type S_4) sites in a strong motion environment, there is concern about the possible occurrence of extensive damage to structures located on such clay deposits due to resonant response of the type observed in Mexico City. For this reason, special dynamic analysis procedures have been adopted for certain classes of structures judged to be susceptible to such damage (see Section 106.2.4). The purpose of the requirements of this section is to define the conditions under which these special analysis procedures are to be applied.

From evaluation of the observed damage in Mexico City, it is judged that damage to structures on Type S_4 soils is unlikely if the natural period of the structure is short or long compared to that of the site. Therefore, structures with natural periods less than or equal to 0.7 seconds have been excluded as candidates for these special analysis procedures. Dynamic analysis is already required for buildings taller than 240 feet, therefore this condition does not change the requirement for dynamic analysis.

C104.8.4 ***General Commentary on Methods of Analysis and Related Structural Models (refers to Requirements Sections 104.8 and 106.3).*** Since the writing of the previous version of these Requirements, there have been significant advances in the development of analytical procedures and in their availability to the design profession. Specifically, programs for analysis of three-dimensional structural models for frames, braced frames, and shear wall combinations are commonly available and widely used for both static and dynamic loading conditions.

However, it is important to recognize that the manual analysis methods, such as assumed inflection point locations, portal method, cantilever method, moment coefficients, etc., still have an essential role. In many cases they are adequate for final design. In other cases they are useful for preliminary design, initial member sizing, and the approximate checking of the results from more detailed analytical procedures. Equilibrium checks of the static lateral force and individual mode dynamic response results are also valuable.

C104.8.5 ***Choice of Two- or Three-Dimensional Structural Models.*** Current professional practice and computational capabilities may lead to the choice of the three dimensional model for both static and dynamic analyses. Although three-dimensional models are not specifically required for static force procedures, nor for dynamic analysis procedures for structures having only vertical irregularity, they often have important advantages over two-dimensional models.

A three-dimensional model is appropriate for the analysis of torsional effects (actual plus accidental), diaphragm deformability, and systems having nonrectangular plan configurations. When the three-dimensional model is needed for any purpose, it can also serve for all required loading conditions, including seismic loading in each principal direction, other selected directions, and for orthogonal effects.

C104.9 System Limitations

It is recognized that the Requirements of this document are based on certain assumptions regarding the configuration of the structure and performance of members. Since these assumptions may not always be justified, the engineer may perform more exact analyses. Care should be taken to ensure that all relevant phenomena have been considered. Requirements of this section shall not be used for the purpose of reducing the prescribed requirements.

C104.9.1 ***Discontinuity.*** While this section addresses the weak story problem in translational directions, it is also important to detect and correct conditions of weak torsional resistance in any story. For example, a single strong element could result in compliance with the 65 percent story strength requirement, but its location could be such that the particular story has less than 65 percent of the torsional resistance of the story above.

Weak stories have been one of the principal causes of structural failures in past earthquakes. For structures with a weak story (less than the 80 percent limit) no mitigating requirements are required (other than the limits resulting from the classification as an irregular structure) unless the story strength is less than 65 percent of the story above. It should be recognized that the allowed 65 percent limit is quite liberal and is meant to give the engineer some flexibility in handling difficult architectural configurations. However, the engineer should verify that the permissible level of story strength variation does not lead to a collapse mechanism under the maximum level of ground motion. The 80 and 65 percent limit values were assigned by judgment; and, as such, are subject to future modifications as experience develops from the use of these Requirements.

The exception, where a weak story is allowed if the members and elements are capable of resisting $3(R_w/8)$ times the design force, is intended to ensure that the structure can survive the force levels induced in the inelastic structure by the maximum expected level of ground motion. The capability to resist may be evaluated on the strength design basis for reinforced concrete, and at a factor of 1.7 times the basic allowable stress resistance for other materials.

These weak story Requirements are intended to avoid concentration of excessive inelastic deformation and behavior at one level, which could result in possible collapse. This concept should be applied to the entire structural system, e.g., abrupt reductions in lateral and/or torsional capacity should be avoided. When resistance requirements at a level allow framing, bracing, or wall elements to be reduced, this reduction should be uniformly distributed over several levels to preclude the possibility of concentrated inelastic deformation at one level.

C104.9.2 ***Undefined Structural Systems.*** New or unusual structural systems that have not had a proven good performance record in seismic events should be evaluated with special care to ensure that they can perform according to the objectives of this document.

An investigation of both the elastic and inelastic deformation demands in the system and in the individual elements should be made at the level of maximum ground motion. This should include consideration of the possible differences between the response of the all elastic model and the response of the inelastic structure. It is suggested that the story and structure yield mechanisms of the structure be determined as a part of this investigation. Some guidelines are given in *Seismic Design Guidelines for Essential Buildings* [U.S. Army, 1986].

Use of any new system proposed for regions of strong seismicity presents special concerns. It is not sufficient to show only strength capability at an appropriate design level. There must also be special details for the inelastic behavior of the system, along with a verification of stability at deformations corresponding to the maximum expected ground motion. Additionally, there must be provisions for redundancies, or backup systems, to provide for extra unknown deformations due to uncertainties in structural response and ground motion. For example, for the defined systems: the Requirements specify that special concrete columns must be designed not to have yield hinges and shear failures; concrete shear walls require special boundary elements when highly stressed; steel bracing requires special strength for connections and protection against fracture and excess inelastic buckling. In other words, there must always be something to support vertical loads even when maximum expected deformations are exceeded. The details used in new systems should be carefully assessed to ensure they provide an equivalent level of performance to that of defined systems.

Exaggerated forms of the defined systems may require use of a reduced R_w value along with special details for the particular configurations of elements and their connections. Examples include, but are not limited to, special moment resisting frames reduced down to a few separated, single bay bents; shear walls, usually either of tilt-up concrete or plywood-stud construction, where large openings create a virtual rigid frame configuration; moment resisting or special moment resisting steel frames where all or most columns in a story are weaker in flexure than the beam elements. In these examples, either the redundancy of the system has been significantly reduced, or the location and mode of inelastic energy dissipation is changed from that considered in the related system as defined in these Requirements.

C104.10 Alternative Procedures

Several alternative design concepts in seismic design, such as seismic isolation, energy dissipation and damping systems, have been developed in recent years and new developments will continue to advance in the foreseeable future. Through the efforts of an Ad Hoc Committee, general Requirements for the design of seismic-isolated structures have been developed based on the current state of practice in seismic isolation (see Section 150 et seq.).

C104.11 Types of Irregularity

C104.11.1 ***Vertical Irregularity.*** Examples of vertical irregularities are shown in Figure C104-2. Some of these irregularities (e.g., Figure C104-2a) are evident by inspection but may not have a significant effect on the seismic response, either in the elastic or the inelastic range. Other irregularities may be difficult to detect in the initial design process but may be created by the addition of nonstructural but stiff infill walls above the first story.

C104.11.2 ***Assessment of Irregularity.*** A quantitative assessment of most of the irregularities shown in Figure C104-2 can be achieved by means of either a story-by-story stiffness evaluation or a deflection analysis of the complete structure. Such an analysis will identify the irregularities shown in Figures C104-2a through C104-2f, possibly that shown in Figure C104-2g, but not those shown in Figures C104-2h and C104-2i. The mass irregularity of Figure C104-2h will, however, have an effect on the modes of vibration and on the distribution of equivalent lateral forces.

C104.11.3 ***Irregular Plan.*** Examples of plan irregularities are shown in Figure C104-3. Such irregularities may arise from nonsymmetric (Figures C104-3 through C104-3e) or even symmetric (Figure C104-3d) plan configurations with re-entrant corners, nonsymmetric distributions of lateral rigidities (C104-3e through C104-3g), nonsymmetric mass distribution (Figure C104-3h), or any combination of these. In most of these cases, torsional modes of vibration may be excited whose effects may not be adequately represented in the equivalent lateral force procedure. The irregularity in the configurations shown in Figures C104-3c through C104-3d is caused primarily by the existence of re-entrant corners that form coupled wings, which may respond in an opening or closing fashion. This may give rise to high stresses in the horizontal diaphragms in the vicinity of the re-entrant corners.

In Figure C104-3, it should be recognized that the configurations a, b, and c are geometric irregularities that can be made to have a regular (nearly uniform translation) deformation behavior by an appropriate distribution of stiffness elements in the plan.

If the structure is irregular in plan only, then it is assumed that the equivalent static force method that includes the accidental eccentricity in mass center locations, will provide a reasonable upper bound on the force distribution and deformations due to torsional mode effects. However, in the case of extreme or unusual configurations, it is advisable to perform a dynamic analysis on a three-dimensional model of the structure.

When the structure has both plan and vertical irregularity, then a three-dimensional model is necessary for the dynamic analysis in order to best evaluate the force distribution and deformations due to the three-dimensional mode combinations for a given direction of seismic input.

C104.11.4 ***Irregular Force Transfers.*** Figure C104-2i shows a severe irregularity in force transfer that would not be identified by a stiffness or deflection analysis. Under the deformation response due to major seismic ground motion, the axial load demand on the exterior column may be much higher than that resulting from specified design loads, and this has necessitated the extra design Requirements in Section 105.7.2.

Irregularities in force transfer, such as those caused by discontinuous columns (Figure C104-4a through C104-4c), discontinuous or offset shear walls or bracing (Figure C104-4d), weak stories in shear and/or torsion, large openings in horizontal framing systems at any level, and the influence of stiff nonstructural elements can cause destructive concentrations of inelastic behavior at the discontinuities. The use of the R_w value for a given structural system assumes that the inelastic behavior is reasonably well distributed throughout the entire structural system. Therefore, if irregularities or discontinuities create concentrations of inelastic behavior, then the irregularities or discontinuities and resulting concentrations of inelastic behavior must either be corrected or strengthened, or the R_w value must be decreased to reflect the limited and concentrated amount of inelastic behavior in the structural system.

If the relative strengths of the adjoining elements are large, inelastic behavior could be concentrated in the shaded elements of the systems shown in Figure C104-4. The column in (a), the support girder in (b), the cantilever beam segment of (c), and the columns and transfer diaphragm in (d) all could experience high force levels ranging from about a $3(R_w/8)$ multiple of specified forces to a nearly fully elastic response forces. This assumes there is no significant yielding in the other portions of the structure under major earthquake input.

The weak story condition is defined in Table 104-4 (see Requirements), Irregularity Type E. A weak story is a discontinuity in capacity that may result in inelastic behavior being concentrated in the particular story. This can cause subsequent deterioration and failure of vertical load bearing elements in a major level of earthquake ground motion. Examples are:

1. A story that has a weak column/strong beam condition at all or most columns, with a strong braced frame in the stories above.
2. A bearing wall system that has many openings and slender wall piers in the weak story with few or no openings in the walls of the stories above.

In the first example, a yield mechanism in the weak story could form, deteriorate, and develop sufficient P-delta effect to lead to collapse. In the second example, the wall piers in the weak story could suffer shear and/or flexure damage, leading to loss of vertical load-carrying capacity.

The strength comparison method used in the definition of Type E in Table 104-4 (see Requirements) is intended for cases such as the foregoing examples. If, however, the system is a strong column/weak beam frame, or coupled shear walls where inelastic behavior can progress away from a story into the upper story beam connections or wall spandrels, then the story strength comparison must consider the particular highest stressed elements that may initiate inelastic behavior. It should then be determined if a story mechanism could occur.

One of the most impressive actual cases of a weak story failure was the Olive View hospital in the 1971 San Fernando earthquake. The structural system was a moment resisting concrete frame supporting a rigid concrete box wall system. Subsequent design requirements have virtually prohibited this type of mixed system: specifically, concrete SMRFs must have strong column/weak beam conditions; and in mixed systems the R_w value of the system in any story must not be greater than that of the system above the story.

The specific irregularity Type E and related Requirements in Section 104.9.1 are intended to prevent system configurations that would have concentrations of inelastic deformations within a story and lead to life-endangering damage or collapse of that story. The object of corrective (extra strength) provisions is to force inelastic behavior beyond the weak story, preferably into other portions of the lateral force resisting system and away from the primary vertical load-carrying elements.

It is important that the engineer understand the intent and objective of weak story requirements, which is to prevent localized inelastic deterioration leading to serious damage or collapse. The simple comparison of story strength, the irregularity penalties, and the extra strength requirements are intended to alert the engineer to the fact that there may be a serious deficiency in the system and to give minimum corrective measures that must be taken. There may be some cases where these corrective measures are not sufficient and better design is left to the judgment and expertise of the engineer. Therefore, whenever the comparative story strength or configuration of a system indicates the possible existence of a weak story, the engineer should proceed to investigate and provide for extra strength in the apparent weak story and/or provide alterations in the upper stories to force inelastic demand into the other stories. Special detailing is, of course, required if significant inelastic behavior is indicated in the weak story.

Seismic Design Guidelines for Essential Buildings [U.S. Army, 1986] provides a procedure for estimating inelastic demands due to the major level earthquake (the EQ-II described in *Seismic Design Guidelines*).

C105 UBC §1628 Minimum Design Lateral Forces and Related Effects

C105.1 General

In the equivalent lateral force procedure, the time-varying inertia forces are replaced by equivalent static forces applied at the story levels. The relative magnitudes of these equivalent story forces are based on simplifying assumptions for mode shapes and modal participation. For the equivalent lateral force procedure, it is assumed that the story drift ratios and masses are reasonably uniform over the height of the building, and that the governing modes of vibration are primarily translational, not torsional.

C105.1.1 and C105.1.2 It should be noted that when orthogonal effects are to be considered, as specified in Section 108.1.3 (see Requirements), the requirement to consider the random direction of seismic input may be assumed to have been satisfied.

The direction of earthquake ground motion is random. It bears no relationship to the axes of the structure. A building's response to earthquake ground motions is a composite response of concurrent motion in relation to the several axes about which the structure can vibrate. Normally, this will include translational motion about two principal orthogonal axes in plan, plus torsional motions.

Nevertheless, for buildings, it is implied that the independent design about each of the principal axes will generally provide adequate resistance for forces applied in any direction. One cautionary note is worth emphasizing: exterior and re-entrant corners of buildings are especially vulnerable to the effects of concurrent motions about both principal axes. The engineer should pay particular attention to the effect of combined forces on members common to systems along both axes.

For structures other than buildings, the requirement that the earthquake force be considered to come from any direction should be respected more rigorously. For structures circular in plan such as tanks, towers and stacks, the design should be equally resistant in all directions.

Response effects due to vertical components of ground motion are not usually calculated. They are considered to be accommodated in the difference between design and actual vertical loads and in special requirements relating to reduced dead load where such reduction may produce important effects. Also, see Section 105.10 (see Requirements).

C105.1.3 For the purpose of designing individual members of a floor system for vertical loads, a 20 psf load is used to represent localized partition loads. However, since this 20 psf value does not occur at all locations at the same time, a 10 psf load is used to represent the average weight of partitions for seismic load evaluation. This load is appropriate for normal partitions; other cases may require higher values. In determining the seismic dead load to be used, consideration should be given to the duration of snow and live loads. When these loads are expected to act for a significant length of time, they should be included in the seismic dead load used to determine seismic forces. Since these loads may not always be present, they shall not be used in the calculation of period according to Method B.

For snow loads, in addition to duration, consideration should be given to roof configuration and distribution of loads. When roof configuration and/or siting is such that the snow could be shaken from the roof in a seismic event, a large reduction may be justified.

C105.2 Static Force Procedure

C105.2.1 ***Design Base Shear.*** The design base shear (Equation 105-1; see Requirements) provides the level of seismic design forces for a given structural system. The resulting force level is based on the assumption that the structure will undergo several cycles of inelastic deformation during major earthquake ground motion, and, therefore, the level is related to the type of structural system and its estimated ability to sustain these deformations and dissipate energy without collapse. The force level determined by Equation 105-1 is used not only for the static lateral force procedure, but also as the lower bound for the dynamic lateral force procedure under Section 106. Therefore it is appropriate to discuss the physical relationship between the design base shear Equation 105-1 and the spectral representation of major earthquake ground motion, as well as the idealization of the inelastic behavior of the structure due to this ground motion. Equation 105-1 provides an explicit relationship between the base shear and the following factors: the spectral representation of the maximum expected ground motion, ZC, the system quality factor, R_w, and the occupancy importance factor, I. The resulting seismic forces are for the allowable stress design basis. This explicit relationship is important because it identifies the relative magnitude of expected inelastic behavior, as reflected by the R_w value.

The I value raises the level at which inelastic behavior may occur, and thereby increases the level at which the function or operability of critical or essential facilities is compromised.

The ZC portion of the base shear equation is an engineering representation of the site-dependent earthquake ground motion having a 10 percent probability of exceedance in 50 years. When multiplied by the gravity constant g, it may be considered as a multi-mode, effective acceleration response spectrum envelope. As such, it may be used to evaluate the forces and displacements induced in a linear elastic structure when this structure is subjected to the maximum expected ground motion. Equations 105-1 and 105-2 (see Requirements Section 105.2) are modified forms of Formulas 4-1 and 4-2 given in the ATC 3-06/NEHRP [ATC 3-06, 1986]. The constants have been adjusted to provide the nearest equivalent design forces for the allowable stress basis used in these Requirements.

It would in most cases be economically prohibitive to design buildings to remain elastic for all levels of earthquake ground motions. A fundamental tenet of seismic design is that inelastic yielding is allowed to accommodate seismic loadings as long as such yielding does not impair the vertical load capacity of the structure. In other words, damage is allowed in the maximum expected earthquake loading case only if it does not pose a significant probability of the structure's collapse. The utilization of design based on linear analysis methods is reconciled with the allowance of damage from inelastic response by using base shears for linear design that are reduced by a factor $1/R_w$ from those that would be expected to occur in the fully elastic structure. Thus, the base shear used in design at the allowable stress basis is $(1/R_w)(IZCW)$. R_w represents the ability of the structural system to accommodate loads and dissipate energy in excess of its allowable stress limit without collapse. The reduction in force is comparable to previous Requirements where the K factor was used. An extremely brittle structure requires a low R_w value (approaching 1), while a very resilient, ductile system can have a fairly high value ($R_w = 12$). Thus the greater the capacity of the structure to accommodate deformations through yielding and inelastic energy dissipation, the higher the value of R_w.

These Requirements are primarily intended to provide life safety, not property protection at the maximum expected earthquake level. Thus, R_w values are assigned so that *each* system is expected to provide the same degree of life safety in the case of the maximum expected ground motion. Many factors contribute to the actual selection of the value of R_w, as discussed below and in Commentary Chapter 1, Appendix C. Among these is the degree of certainty in performance at limiting loads. When two systems have similar mean expected performance, but one has a greater degree of uncertainty in performance, then that system is assigned a lower value of R_w. In this way, comparable reliabilities of noncollapse are maintained.

Observations of structural systems responding in the inelastic range indicate that as the structure yields, the period, damping, and other dynamic properties change, often substantially. The effect of these changes is that while the force levels actually experienced in the structure are greater than those employed in design ($ZICW/R_w$), they are less than those that would occur in a fully elastic response (ZCW), and the displacements may be larger. The more ductile the system's performance, the greater is its capacity to accommodate inelastic displacements and forces. These relationships are not easily represented; however, to a first order, they are proportional to R_w, a coarse representation of the inelastic energy dissipation capacity of the structural system.

Design provisions and related material detailing methods recognize the energy dissipation capacity requirement, as represented by R_w, and are intended to ensure that this inelastic behavior can take place without major failure or collapse of the structural system. For a given structural system having a design level as determined by the R_w value, the following factors could produce a structure having a total yield mechanism resistance significantly larger than the specified design base shear value. Contributors to seismic resistance are:

1. Working stress design basis
2. Multiple load combinations

3. System redundancy
4. Participation of other structural and nonstructural elements in resisting lateral forces
5. Strain hardening
6. As-built member configurations

As a result of these combined "overstrength" contributions to the lateral force versus deformation behavior of the structure, individual members and elements can be subjected to forces significantly larger than the specified design force level when the structure accommodates the inelastic deformations caused by the maximum expected level of ground motion. The extent of the increases was judged by SEAOC to be about 3 for a building frame system with shear walls or concentric bracing ($R_w = 8$ for systems B3 and B4 of Table 104-5): therefore, the increase factor was selected as $3(R_w/8)$. Note that the incorporation of the R_w value in this overstrength factor makes the corresponding maximum base shear equal to a common value of $3ZC/8$ for all systems.

Critical elements required for the stability and integrity of the structure, such as columns under discontinuous shear walls or bracing systems and other elements in irregular force transfer paths, must have the strength capacity to resist the $3ZC/8$ force unless the yield capacity of adjacent elements is less than this level. The $3(R_w/8)$ factor is therefore applied to the design of elements and connections whose yield or failure could result in local or general collapse (see Section 105.7.2 for discontinuous columns; Section 104.9.1 for weak stories; and Chapter 7 for related steel column, bracing, and connection requirements). The load combinations and capacity requirements involving the $3(R_w/8)$ factored seismic loads apply only to the specified structural elements and their connections to the adjoining elements. The adjoining elements need not be designed for the forces due to these factored loads. It is intended by these Requirements that forces resulting from load combinations involving the $3(R_w/8)$ factored seismic loads may be resisted on a strength basis.

Also, in order to ensure that inelastic energy dissipation can occur in the intended elements of the lateral force resisting system, it is recommended that collector elements and their connections, and superstructure to foundation connections have the strength to resist the load combinations in Section 105.7.2.

Similarly, structural deformations will be at least equal to $3(R_w/8)$ times the design level deformation. Here, the $3(R_w/8)$ factor is a direct carryover of the $3.0/K$ factor from previous editions of these Requirements and is therefore based more on tradition than on hard facts. Elements that are not part of the lateral load resisting system and that are essential for the vertical load-carrying system, must be capable of tolerating these deformations without failure (Section 108.2.4). This factor is also used for determining movement of exterior elements (Section 108.2.4.2), building separation limits (Section 108.2.11), and eccentric brace link beam rotation limits (UBC §2211.10.12).

Nonelastic deformations due to the maximum expected earthquake ground motion can be quite variable because of particular patterns of structural yielding, equivalent damping of the yielding elements, and changes in the stiffness and period of the structural system as yielding occurs. Therefore, in some cases, the maximum deformations can actually exceed the factored values, and could be on the order of R_w times the design level deformations; critical elements (particularly columns) should have ductile detailing capable of accommodating greater deformations than indicated by the factor $3(R_w/8)$. In summary, for structural systems at their fully deformed state due to the maximum expected ground motion, the force amplification of $3(R_w/8)$ is generally on the high side and includes some inherent load factor; while the deformation factor of $3(R_w/8)$ is on the low side. The equality of these two "consensus" factors should not be construed as meaning that the structure has a linear behavior at its maximum deformation state. A better representation of the inelastic deformation may be achieved by an analytical model that employs the more flexible conditions of the structure, such as noncomposite sections for steel, and cracked sections for reinforced concrete.

When R_w is less than 6 for systems in Tables 104-6 and 104-8, the factor $3(R_w/8)$ may become smaller than the engineer may want to use in design. In specific cases where the engineer has reason to be concerned, values less than 2 for this factor may be inappropriate. The factor is used herein to accommodate the increased loading and deformations caused in the inelastic structural response. Further, it is meant to provide an added level of assurance of structural stability for regions, such as Zones 2 or lower, where the large earthquakes that can occur are substantially larger than those with a 10 percent probability of exceedance in 50 years.

C105.2.1.1 *Discussion of Factors in Base Shear Equation 105-1*

1. **The Seismic Zone Factor Z and Table 1-A.** The seismic zone factor Z represents the effective peak ground acceleration (EPA) corresponding to the maximum expected level of earthquake ground motion for each Zone listed in Table 104-1 and defined in Figure 104-1. The factor Z is expressed as a fraction of the gravity constant g; and as an EPA value it is used to scale the spectral shape represented by C (Equation 105-2), so that the product ZC represents an acceleration response spectrum envelope (as a fraction of gravity) having a 10 percent probability of exceedance in 50 years (see Appendix 1A to Commentary Chapter 1, "Seismic Zone Factor," for additional Commentary).
2. **The C and S Coefficients.** The curve given by Equation 105-2 is a simplified multi-mode envelope acceleration response spectrum normalized to an EPA = 1 basis. The included site coefficient S adjusts the shape to the appropriate frequency response content for the site soil characteristics. In comparison with the response spectrum shapes given in Figure 104-2, which vary with the reciprocal of T, the C of Equation 105-2 uses the $^2/_3$ power of T to provide a reasonable upper bound value of multi-mode response (see Appendix 1B to

Commentary Chapter 1, "C and S Coefficients," for further related Commentary).

3. **Maximum Value of the C Coefficient.** A maximum limit on $C=2.75$ for any structure and soil site condition is given to provide a simple seismic load evaluation for design projects where it is not practical to evaluate the site soil conditions and the period of the structure.
4. **Minimum Value of C/R$_w$.** The lower limit of $C/R_w=0.075$ is intended to ensure that a minimum base shear of 3 percent of the building weight is used in Seismic Zone 4, with proportional values in the lower zones. In consideration of our present uncertainty in forecasting ground motion and structural response, it was judged prudent to establish this minimum design requirement until more experience and knowledge is available for long period structures subjected to major levels of ground motion.
5. **Importance Factor I and Table 104-3.** The I factor for essential facilities has been reduced from 1.5 to 1.25 and new classifications of hazardous facilities and special occupancy structures have been added. An I_p factor for elements, components, and equipment has been added.
6. The reduction in force level is made recognizing that higher force levels alone do not necessarily improve seismic performance. Experience indicates that independent design review, an appropriate program of testing and inspection, and involvement of the engineer in the construction support process will result in a higher standard of structural performance. Therefore, the I factor reduction to 1.25, coupled with these new Requirements, are judged to achieve this purpose more reliably and economically. Also, many engineers believe that specific damage control measures are a better means of achieving improved performance in lieu of the I factor (see Commentary Section C201).
7. **Structural System Factor Rw and Table 104-6.** For a given structural system and material, the R_w value is a measure of the ability of the system to sustain cyclic inelastic deformations without collapse. As such, it enters into the denominator of the design base shear (Equation 105-1), so that design loads decrease for systems with large inelastic deformation capabilities. The R_w system factor has been introduced to provide a physical concept of system performance (R_w) for use in the base shear format. The simple relationship of $R_w=8/K$ is applicable to the previously used classifications of systems ranging from bearing wall to special moment resisting frames. Both new knowledge on performance and the development of new systems required the more extensive and detailed description of systems and assignment of R_w values as given in Table 104-6. A discussion of the methods and concepts involved in these assignments is given in Appendix 1C to Commentary Chapter 1, "R_w Factor."

C105.2.2 *Structure Period*

C105.2.1 ***Method A.*** The form of the period Equation 105-3 was taken from the ATC 3-06 document, where it was originally applicable only to moment resisting frame systems. The modifications make it applicable to systems with shear walls and bracing elements.

Moment Resisting Frame Systems. The use of total height h_n was judged to be more appropriate than the number of stories N, as used in the frame period equation given in the previous editions of these Requirements. Many building configurations tend to have considerable variations in story heights, therefore the number of stories is not a good indicator of period. Where ATC 3-06 originally gave values of C_t equal to 0.035 and 0.025 for steel and concrete frames, respectively, these Requirements give 0.035 and 0.030. The data upon which the ATC 3-06 values were based were reexamined for concrete frames and the 0.030 value judged to be more appropriate

The values for C_t given in these Requirements are intended to be reasonable lower bound (not mean) values for structures designed according to the Requirements given herein. Recent surveys and studies of the particular buildings that provided the period data for the ATC 3-06 equations [Bertero et al., 1988] have shown that these original equations, even with the modified C_t for concrete, provide predictions that are about 80 to 90 percent of the lower bound values of measured periods at deformation values near first yield of the structural elements. While this might indicate a large, perhaps excessive, degree of conservatism, it is most important to recognize that the buildings involved were designed for lateral force level requirements prior to those of the 1976 UBC, which are significantly lower than those in the 1976 UBC, as well as those given in these Requirements. Furthermore, the controls on interstory drift for all elements, on irregularity and member detailing provisions, as given in these Requirements, are generally more restrictive than those used for the buildings in the period evaluation study. Therefore, given reasonably similar nonstructural elements, the population of structures that conform to the drift provisions of these Requirements will have increased stiffness and corresponding lower period values than the structures designed according to previous codes.

It is recognized that the values for C_t provide period estimates that are lower than most measured values in the elastic range, and definitely much lower than nearly all measured values in the cracked section state for concrete buildings and the partially yielded state for steel buildings. However, these estimated period values for each material, when used in the C factor, provide design values that SEAOC judges to be appropriate and consistent with past design experience. For the usual case of a descending spectrum, the decrease in demand due to the increase in period as the structure deforms into the inelastic range is already included in the R_w value of a given structural system (Figure App1C-1). Therefore, period formulas should provide the period of the structure in its elastic state. Improved formulas, similar to

Equation 105-4, need to be developed to represent the type, configuration, and amount of lateral force resisting elements, as well as other building characteristics, that govern the period value. However, until these relationships are available, the engineer may employ Method B in order to better incorporate the building properties in the evaluation of the period.

Lateral Force Resisting Systems With Shear Walls. The format of Equation 105-3 has been adapted to represent the configuration and material properties of these systems. The corresponding equation in the previous Requirements considered only the gross building dimensions (rather than the amount and configuration) of the shear wall elements, and as a result had wide discrepancy between real and computed period. A reasonable agreement with available building period data is achieved with the modification of the C_t coefficient by the A_c value given by Equation 105-4. In this equation, the summation is with respect to the index "e" for all the shear walls having their planes parallel to the direction of the applied forces. A method of defining A_c and C_t for braced frames has yet to be developed. The use of the 0.02 value for C_t does not specifically represent the bracing configuration; the engineer may wish to consider use of Method B.

Further discussion of the development of Equation 105-4 is given below. It will be possible to better calibrate this equation and its coefficients as more data become available.

For buildings with other than moment resisting frames, the previous edition of these Requirements and the 1985 UBC determine the approximate period from the following equation

$$T = 0.05 h_n / \sqrt{D} \qquad \textbf{Eqn. C105-1}$$

During the development of approximate period formulas for the ATC 3-06 project, the measured periods of a number of buildings were obtained. These measured periods were used to establish the revised equation for moment resisting frames in ATC 3-06 and for these Requirements and the 1988 UBC. There was a proposal made during the development of ATC 3-06 to substitute L_s, the length of the longest element of the vertical resisting system, for D in Formula 1-3A of the previous edition of these Requirements. This obviously does not account for the stiffness contribution of all of the other vertical resisting elements in the system.

The stiffness of a shear wall is inversely related to the deformation of the wall, and the deformation consists of shear and flexural contributions. The shear deformation is governed by the wall area, $D \times t$. The flexural deformation is governed by the moment of inertia of the wall.

The new Equation 105-4 in these Requirements for shear wall structures was developed using some simplifying assumptions. The impetus for this effort resulted from the difficulties encountered in the trial design effort for the ATC 3-06 document. In regions where wind loads should be larger than seismic design loads, reasonable calculated estimates of the true period of the structures (as governed by wind design) were several times larger than the periods given by the approximate equation. The approximated (short) period values gave seismic design loads erroneously in excess of the wind load requirements. By the same token, Formula 1-3A, as set forth in editions of these Requirements prior to the 1990 edition, can in some cases overestimate the calculated period of a shear wall structure. Hence, this formula was revised based on available data for shear wall buildings. The revised equation (Equation 105-4 in Requirements) yields reasonable agreement between calculated and observed period values.

The approximate period for "all other structures" is to be determined from Equation 105-3 in the Requirements.

The formulae presented for approximate period are appropriate only for structures with rigid diaphragms. Structures, especially lowrise structures with flexible diaphragms, will generally have periods related to the stiffness of the diaphragm rather than the stiffness of the vertical resisting elements. There are, at present, no proposals for considering this phenomena in determining the approximate periods for such structures.

C105.2.2.2 ***Method B.*** Method B permits the evaluation of period T by either the Rayleigh type of Equation 105-5 or by any other properly substantiated analysis. If a mathematical model has been formulated for the dynamic analysis of Section 106, then the period of the first mode of vibration in a given principal direction can be used for this T value.

The period, T, as determined by Method B, cannot be more than 30 percent greater than the period determined by Method A (Equation 105-3) in Seismic Zone 4 and 40 percent greater in other zones. Reasonable mathematical modeling rules should be followed such that this increase in period is not obtained when the actual structure does not merit it. Some suggested rules are given in the Commentary to Section 106.3. Although it would have been preferable to specify modeling rules in these Requirements, rather than impose the 30 percent and 40 percent limit, it was not possible to obtain a SEAOC consensus on a complete set of rules to meet all the possible variations and conditions encountered in design. Note that the period limitation results in a base shear from Method B that must be at least 84 percent of that determined using Method A ($(1/1.3)^{2/3} = 0.84$) for structures in Seismic Zone 4 and 80 percent for all other zones.

C105.2.2.3 ***Special Period Evaluation for Top Force F_t.*** When Method B is used to determine period, the value of T used to determine the top force, F_t, is subject to the same period increase limitations of 30 percent in Seismic Zone 4 and 40 percent in all other zones.

C105.3 Combinations of Structural Systems

C105.3.1 ***Vertical Combinations.*** The provision requiring the smallest R_w is intended to prevent mixed systems that may have concentrations of inelastic behavior in the lower stories. This type of behavior results from putting a stiff shear wall system, $R_w=6$ or 8, on top of a SMRF system, $R_w=12$ (e.g., Olive View hospital). Note that this concentration of inelastic behavior can also result from using a bearing wall, $R_w=6$, or a braced frame, $R_w=8$, structural system on top of a steel OMRF with an $R_w=6$.

This section also has Requirements for designing structures with certain setback configurations such that the structure may be treated as two independent structures—a tower and a platform or base. The procedure set forth in Section 105.3.1.2 is also applicable for structures having the same R_w values for both their tower and platform portions.

C105.3.2 ***Combinations Along Different Axes, Item 1.*** This provision is intended to limit the nonelastic yielding deformations that may occur perpendicular to the bearing wall. The vertical load capacity of the bearing wall system may be destroyed if this orthogonal deformation is not controlled. The requirement for an equal R_w value is for determination of the design load level to be used for the orthogonal system; the details of the orthogonal system should conform to those required for the actual orthogonal system's R_w value.

C105.4 Vertical Distribution of Force

Equation 105-6 is based on a linear mode shape, and the F_t in Equation 105-7 (see Requirements) is to account for higher modal effects at the top of the structure.

When the structure differs significantly from the assumed linear mode behavior, consideration should be given to alternate methods of distributing the forces such as those described in Section 106.

C105.5 Horizontal Distribution of Shear

Consideration should be given in the modeling of structures where vertical resisting elements are tall enough to have significant flexural deformations. The rigidity of these elements can be substantially different from rigidities calculated by the simple story-by-story method. Also, considerable differences in the distribution of shears can occur when the diaphragm deformations have been considered, compared to results obtained for fully rigid diaphragms.

The location and distribution of all of the weights required to be considered for earthquake motion response cannot be determined with complete certainty. The Requirement for displacement of the mass at each level was introduced to account for this condition, along with others listed in Commentary Section C105.6, below. The Requirement applies to all structures, whether the diaphragms are flexible or rigid.

C105.6 Horizontal Torsional Moments

When the diaphragm deformation is more than two times the associated story drift, then the diaphragm is considered to be flexible. Here the term "flexible" implies that the diaphragm may be modeled as a simple beam between vertical resisting elements. When the diaphragm deformation is less than or equal to twice the story drift, then the Requirements of this section apply. In most cases, the diaphragm may be modeled as "fully rigid" without in-plane deformability. However, there are structural configurations—such as vertical resisting elements that have large differences in stiffness or offsets between stories, and diaphragms with irregular shapes and/or openings—where the engineer should investigate the effects of diaphragm deformability. The use of the most critical results obtained from the "fully rigid" and the "flexible" models would be acceptable.

The accidental horizontal torsional moments are specified to account for:

1. Differences between the analytical model and the actual structure
2. The real nonuniform distribution of both dead and live load
3. Eccentricities in the structural stiffness due to nonstructural elements such as stairs and interior partitions, and/or
4. Torsional input loading on the structure due to differences in the seismic ground motion over the extent of the foundation

For example, traveling wave effects can induce torsional motions in structures having a long plan dimension [Werner et al., 1979]. Furthermore, when such structures are located close to the causative fault, the two components of horizontal ground motion in this near fault region can be significantly different, and this condition can accentuate the torsional motions.

The eccentricity amplification factor A_x is intended to represent the increased eccentricity caused by the yielding of perimeter elements. This factor provides a simple yet effective control on systems that might otherwise have excessive torsional yield in a given story. These Requirements are intended to avoid potential torsional failure mechanisms by ensuring that the structure has the stiffness and the strength to resist both calculated and accidental torsion effects. In the calculation of A_x by Equation 105-9, the value of δ_{avg} should be found from the displacements resulting from the same torsional moment used to evaluate δ_{max}.

Torsional analyses using the amplified accidental torsion should be repeated until Equation 105-9 results in an A_x value that is less that or equal to the preceding A_x value used to evaluate the displacements.

The statement "The most severe load combination for each element shall be considered in design" is intended to permit torsional shear to be subtracted from the direct load shear when the torsional shear is opposite to the direction of the seismic load. In that case, the torsional shear must be due to the smaller torsion that is reduced by the accidental torsion requirements. For the case where the torsional shear is in the same sense and direction of the direct load shear, then the more severe loading would be due to the larger torsion that is increased by the accidental torsion requirements and shear added to the direct load.

C105.7 Overturning

C105.7.1 These Requirements for stiffness and strength are intended to prevent local failure or collapse in elements provided either for redistribution or at discontinuities in the lateral load resisting system. Elements used to redistribute or transfer the effects of overturning moments should be designed so as to have either:

1. Special details compatible with those of the elements adjacent to the discontinuities; *or:*
2. Strength capacity necessary to develop the strength of the adjacent elements; *or:*
3. Capacity to resist the loads due to maximum level of ground motion, at least $3(R_w/8)$ times the seismic design load.

C105.7.2 While this provision pertains specifically to columns, it is recommended that similar resistance and ductility requirements be applied to transfer girders (Figure C104-4b), and girders with offset columns (Figure C104-4c), where these girders support columns carrying axial seismic loads.

C105.8 Story Drift Limitation

C105.8.1 It is most important to recognize that the drift limitation applies to the location of any vertical support or resisting element in the plan of a story, and therefore is more restrictive than a control on average story drift. For buildings with period values equal to or greater than 0.7 seconds, the specified drift ratio limit of $0.03/R_w$ is slightly more stringent than the equivalent ratio ($0.005\ K$ corresponding to $0.04/R_w$) given in the previous editions of these Requirements because for most lateral force systems, the base shear is slightly less than that used in previous editions.

C105.8.2 Exceeding the stated drift limits should be approached with caution and with an understanding that major earthquake ground motion can cause deformations substantially larger than those resulting from design forces. The $3(R_w/8)$ multiplier of the design deformations provides an estimate of these major deformations in other sections of these Requirements. In some cases, including near-source ground motion, and/or irregular structural configurations, this estimate may be low, and the engineer should evaluate the consequences of larger deformations as they relate to the design objectives. The engineer must also consider the related drift or displacement control Requirements of Section 105.2 for the separation of a structure adjacent to another structure (see Section 108.2.11). Also, see Commentary Section 105.2.1.

C105.8.3 The permission to use the drift values resulting from the *C* value obtained by Method B of Section 105.2.2.1.2 provides an incentive to employ this analytical approach. If the modeling rules given in Commentary Section 107.3 are followed, the resulting Method B value for period *T* and the deformations resulting from the use of the corresponding *C* value are realistic representations of actual structural behavior. The "30 percent and 40 percent of the *T* value of Method A" is a limiting provision for seismic design loads only. With this provision, a single Method B analysis could be performed for the valuation of drift, and the corresponding forces could be scaled to conform to the 84 or 80 percent level of Method A for the required design load values.

C105.9 P-delta Effects

The drift ratio value of $0.02/R_w$ serves to define the threshold of deformation beyond which there may be significant P-delta effects. Further analysis is necessary to determine if the actual combination of vertical load and lateral deformation is cause for concern. The P-delta effects in a given story are due to the horizontal offset or eccentricity of the gravity load above that story. Referring to Figure C105-1, if the story drift due to the lateral forces prescribed in Section 105 were Δ, the bending moments in the story would be augmented by an amount equal to Δ times the gravity load above the story. The ratio of the P-delta moment to the lateral force story moment is designated as the stability coefficient as given by Equation C105-2 in Figure 105-1. This coefficient may also be considered as the ratio of the increased shear demand $P_x\Delta/h_{sx}$ to the design story shear V_x. If the stability coefficient θ is less than 0.10 for every story, then the P-delta effects on story shears and moments and member forces may be ignored. If, however, the stability coefficient θ exceeds 0.10 for any story, then the P-delta effects on story drifts, shears, member forces, etc., for the whole building should be determined by a rational analysis.

Stability coefficient θ is given by:

$$\theta = \frac{P_x\Delta}{V_x \bullet h_{sx}} \qquad \textbf{Eqn. C105-2}$$

where Δ = design story drift

V_x = seismic design shear force acting between level x and level x-1

h_{sx} = story height below level x

$$P_x = \sum_{i=x}^{n} (D+L)_i = \text{ total design gravity load at and above level } x$$

An acceptable P-delta analysis, when required, is as follows:

1. Compute for each story the P-delta amplification factor, $a_d = \theta/(1-\theta)$.a_d, which accounts for the multiplier effect due to the initial story drift, leading to another increment of drift, which would lead to yet another increment, etc. Thus, both the effective shear in the story and the computed eccentricity would be augmented by a factor $1 + \theta + \theta^2 + \theta^3 \ldots$, which is $1/(1-\theta)$ or $(1 + a_d)$.
2. Multiply the story shear V_x in each story by the factor $(1 + a_d)$ for that story and recompute the story shears, overturning moments, and other seismic force effects corresponding to these augmented story shears. Note that the portion of

the augmented shear due to the P-delta effect on columns that are not part of the lateral load resisting system should be transferred by diaphragm action to the lateral load resisting system. For example, if the lateral load system was a perimeter moment resisting frame, the augmented P-delta loads coming from the interior vertical load bearing columns should be adequately transferred by the diaphragms to the perimeter frames.

The columns of moment frames designed with P-delta effects included need not have their bending stresses amplified by the term $(1 - f_a / F_e)$ in AISC Formula 1.6-1a, or δ_s in ACI Formula 10-8, since these factors were intended to account for P-delta effects.

It should be noted that P-delta effects are potentially much more significant in lower seismic zones than in higher seismic zones, because the lateral load resisting systems in higher seismic zones are required to be designed for a larger base shear, V_x, than those in lower seismic zones. Similarly, long period (flexible) structures and structures using high R_x values are more affected by P-delta effects than structures designed for higher base shears.

Some computer programs for frame analysis state that frame P-delta effects are included directly in the analysis. The engineer should verify that the total gravity load employed and the method used in these programs will provide results that are essentially equivalent to the augmented story shear method described herein [Wilson and Habibullah, 1987; Clough and Penzien, 1975]. See also related P-delta Commentary in ATC 3-06 [1986].

C105.10 Vertical Component of Seismic Forces

Design for dead load in members will usually ensure against problems resulting from downward accelerations, while the typical load factors will provide assurance against failure resulting from upward accelerations.

Since cantilevers do not have continuity, it was deemed necessary that some additional assurance against failure from upward and downward accelerations be provided. Also, both simply supported and continuous prestressed beams should be checked for the reduced vertical load combination.

C106 UBC §1629 Dynamic Lateral Force Procedure

This section provides minimum Requirements, limitations, and guidelines for performing dynamic analyses. It is not intended to be a complete coverage and should not be used as a "how-to" manual for the performance of dynamic analysis. The engineer should consult appropriate references on dynamic analysis for detailed procedures [e.g., Clough and Penzien, 1975; Chopra, 1980; U.S. Army, 1986; Biggs, 1964; SEAOSC, 1977].

C106.1 General

Use of the analysis procedures outlined in this section and discussed in Appendix 1D to Commentary Chapter 1 incorporate dynamic aspects of seismic response in the design process. A dynamic analysis, per se, will not necessarily provide response estimates consistent with actual earthquake performance, nor will it give all the answers or solutions to a seismic design problem. The accuracy of the results depends on many things, among which are:

1. The simplifying assumptions made to produce the computational procedures
2. Appropriate assignment of material property values, building dimensions, damping coefficients, and other characterizations of the structure.
3. How close the model reflects the real structure and its foundation conditions.
4. The representation of the maximum expected earthquake input for the site.
5. The correct interpretation of the analysis.

Dynamic analysis can, however, greatly aid the seismic design process by clarifying certain important aspects of the structure's dynamic response characteristics that may not be apparent from the static lateral force procedure of Section 105. For example:

1. The effects of the structure's dynamic characteristics on the distribution of lateral force along its height, which could differ from the linear distribution considered in the static lateral force procedure.
2. The existence of normal modes with significant components of torsional motion that can lead to increased dynamic loads in the structure's lateral force resisting system.
3. The effects of the structure's higher modes of response that could contribute substantially to individual story shears and deformations.

The dynamic lateral force analysis procedures presented in these Requirements were developed to address the fact that the distribution of forces on some structures is often considerably different from those that would be given by the static lateral force procedure of Section 105.2. As an example, such differences can occur in buildings with severe setbacks, buildings of unusual configuration, and buildings with significant variation in drift from one story to another. Implicit in these Requirements on dynamic analysis is the assumption that the analysis will normally be performed using a computer program. Consequently, the Requirements are formulated in a manner suitable for such implementation.

Design force values as obtained from the base shear of $ZICW/R_w$ are employed at the $1^1/_3$ allowable stress level, which corresponds to about 70 to 80 percent of yield strength. Linear elastic analysis can, therefore, be used at this level to obtain member forces and displacements. Since fully elastic structure response could result in a base shear of ZCW, it is evident that structural displacements due to the maximum expected earthquake will be well into the inelastic range. With the controls on structural irregularity given in these Requirements, it is assumed that the displacements in the inelastic structure can be estimated by an appropriate factor times the displacement response of the elastic structure. Therefore, linear elastic analysis may be used for member design forces at load levels below yield, and for member displacements for loads beyond yield. The energy dissipation capability of the structure to sustain these inelastic displacements will be a function of the type of structural system selected from Table 104-6 and the details used by the engineer to accomplish this design. The details and concepts used by the engineer are the critical elements in seismic resistant designs and must be so recognized.

The guidelines of this section provide the engineer with a method for dealing with structures that have characteristics that may violate the assumptions inherent in use of the static approach of Section 105. Regular and symmetric structures generally exhibit much more favorable and predictable seismic response characteristics than do irregular structures; therefore, the use of regular structures is encouraged and irregular structures should be avoided whenever possible.

Implementation of dynamic lateral force procedures involves:

1. Identification of appropriate ground motion representations for use as input to the analysis.
2. Development of an elastic mathematical model of the structure that represents its important geometric, stiffness, inertial, and damping characteristics.
3. Computation of the model's dynamic response to the seismic input motions, using response spectrum or time-history methods and established structural analysis computer programs.
4. Careful interpretation and application of the results of the analysis.

C106.2 Ground Motion

This subsection describes the types of representation of seismic ground motion, principally by a response spectrum, that can be used in a dynamic analysis. The results of a dynamic analysis are normally sensitive to the estimated intensity and frequency content of the seismic ground motion. The spectrum can be considered to have two characteristics: shape and amplitude. For design of structures using these Requirements, the shape of the response spectra is more important than the magnitude because the structural response is scaled as noted in Section 106.4.1.3. The duration of the ground motion, which also is an important factor in building response, is not directly represented by the response spectrum procedures. However, within these Requirements, duration representative of major earthquake ground motion is recognized implicitly by the special Requirements for systems and elements. These Requirements are intended to provide stable levels of resistance under multiple reversed cycles of inelastic deformations.

C106.2.1 ***Normalized Response Spectra.*** The spectrum shapes given in Figure 104-2 are a smoothed average of normalized 5-percent-damped response spectra. They were obtained from actual ground motion records that were grouped by subsurface soil conditions at the location of the recording instrument [U.S. Army, 1986] and are applicable for earthquakes motions characteristic of those that occur in California. Where there are established indications that the site conditions are significantly different from those corresponding to the given standard site-dependent spectra, then a site-specific study should be performed to better define the ground motion: this would apply to cases where the site-specific ground motion can cause significantly different structural response. For site conditions for which the corresponding normalized shapes are appropriate, the ground motion representation having a 10 percent probability of exceedance in 50 years may be taken as the Z-factored normalized shape.

The following equations may be used to represent the normalized spectrum shapes given in Figure 104-2.

Soil Type 1

For $0 \leq T \leq 0.150.15 < T \leq 0.3900.390 < T$

Use $(1+10T)2.5$ $(0.975/T)$

Soil Type 2

For $0 \leq T \leq 0.150.15 < T \leq 0.5850.585 < T$

Use $(I+10T)2.5$ $(1.463/T)$

Soil Type 3

For $0 \leq T \leq 0.20.2 < T \leq 0.9150.915 < T$

Use $(1+7.5T)2.5$ $(2.288/T)$

In order to be more representative of response spectrum shapes in the longer period ranges beyond about 3 seconds, these equations should have a transition to a "constant" displacement relation proportional to $1/T^2$. However, because of the scaling Requirements of Section 106.4.1.3, this modification would not have any significant effect on design forces, and is therefore left for consideration in future revisions of these Requirements.

C106.2.2 and C106.2.3 ***Site-Specific Response Spectra and Time Histories.*** The use of "site-specific" response spectra and time histories are mentioned to allow the engineer to make use of these tools when it is desirable to go beyond the minimum requirements promulgated in these Requirements or where a specific spectrum is required for an S_4 site. For further discussion, see Appendix 1B to Commentary Chapter 1.

The representations of site-specific ground motion should be reflective of all the parameters—such as earthquake source, source-site travel path, (i.e., topography, basins, etc.), and local soil conditions—that influence the ground shaking at a site.

The individual site-specific time histories used should be selected to best represent the regional and local geologic conditions at the site. If the time histories have spectra with significant valleys at periods important to the structure's response, the Requirements of Section 106.2.3 require that the composite spectra from the selected time histories should approximate the site-specific spectra developed in accordance with Section 106.2.2. Recognizing that no recorded time history will match the entire site-specific response spectrum, it is particularly important to select an ensemble of recorded or synthetic motions that best represent the conditions of earthquake source, travel path, and local geologic conditions that exist at the site. The time variation of recorded earthquake motions typically exhibits an initial rise, a central segment of strong shaking, and a decaying tail. The duration of each of these segments is dependent on the nature of the faulting and surrounding geologic conditions. The engineer should recognize this fact and should therefore select site-specific time histories whose time variation is reasonably consistent with the time variation of actual ground motions that have been recorded under comparable geologic conditions.

When time-history analyses are performed, good practice requires that independent time histories be used in each of the orthogonal directions of input and that at least three time histories be used in each direction to ensure that the responses are characteristic of those that may occur, and to provide a sufficient number of response estimates to ensure that the possible response of the structure is adequately represented. Multiple responses provide a basis for estimating the appropriate values of the dynamic forces and deformations in the structure's critical elements.

Again, when complying with these Requirements, the frequency content of the input is important over the full frequency range of the structural response. The magnitude is less important because the final forces must be scaled up, and may be scaled down, to reflect the design levels of the static force procedure of Section 105.

When site-specific spectra or time histories are used, it is recommended that these equal or exceed the level of ground motion having a 10 percent probability of exceedance in 50 years at the site [Schnabel and Seed, 1973; U.S. Army, 1986; Seed and Idriss, 1982]. See also Appendix 1A to Commentary Chapter 1. It is further recommended that the design base shear be determined by the procedure of Section 106.2.4, Item 3, but subject to the minimum design value required by Section 106.4.1.3.1. Note that this minimum value will usually govern when the site-specific spectrum ordinates are not significantly larger than the corresponding values of the spectrum given by the soil type curve of Figure 104-2, scaled by the Z factor. The structural response distribution due to the particular site-specific ground motion characteristics is better represented by the procedure of Section 106.2.4, Item 3. The scaling procedure of Section 106.4.1.3 is intended for use with the spectra of Figure 104-2, which are general averages and not necessarily representative of site-specific conditions.

C106.2.4 ***Structures on Soil Profile Type S_4.*** See Commentary Section C104.8.3, Item 4 for a discussion of the particular site response effects that require special analysis.

Item 2.2. Resonance occurs:

1. When the building's elastic period is close to, but slightly less than, that of the predominant period of the ground shaking.

2. As the building's response becomes inelastic, its period lengthens and coincides with a predominant period of the ground shaking.
3. When the duration of the ground shaking at this period is sufficiently long to allow the development of the amplified building motions that will occur when the building and ground motion periods approach coincidence.

The response of the building is amplified by these resonance conditions, and the resulting increases in the building's earthquake-induced forces and deformations must be considered in its design. The building's amplified response under resonant conditions may be conservatively approximated by using a constant level acceleration response spectrum equal to 100 percent of the maximum acceleration ordinate of the site-specific design spectrum for a damping ratio of 5 percent. The damping ratio is held at 5 percent because any reduced response effects due to the increased damping in the inelastic structure are already included in the R_w factor used in Section 106.2.4, Item 3 to convert the all-elastic response to the design force level. The maximum constant level may be taken from a smoothed average representation of the site-specific acceleration response spectrum.

Item 3. It is likely that the representation of the S_4 ground motion will have unique characteristics that differ significantly from those represented by the spectral shapes of Figure 104-2. Division of the resulting base shear by R_w is consistent with the design philosophy for inelastic design. The result is a design that exhibits a reasonably uniform pattern of inelastic demand throughout the structure. However, the minimum design value required by Section 106.4.3.1 must be met.

C106.2.5 ***Vertical Ground Motion.*** Statistical analyses of ground motion records at distances greater than about 10 kilometers from the seismic source have shown that it is generally reasonable for design purposes to use a vertical component of ground motion whose peak amplitude is two-thirds of the peak amplitude of the horizontal motion. However, for sites located close to active faults, the peak amplitude of the vertical component of ground motion can equal or exceed the peak amplitude of the horizontal ground motion. For such sites, or for any sites with unusual conditions, a site-specific evaluation should be performed to specify vertical motions for seismic design. In all cases, it is important to note that the peaks of the vertical and horizontal components of motion are associated with different wave arrivals and therefore do not occur at the same time (see also discussion under Section 106.4.1.4).

C106.3 Mathematical Model

A mathematical model is an idealized, simplified representation of a complex building structure to facilitate computation of the structure's response for design purposes. Modeling is an art in which the most important characteristics of the structure are synthesized into a mathematical representation for computation of the dynamic response. Regular structures can usually be adequately represented with a one- or two-dimensional model, where accidental torsion is included with additional "hand" calculation. Very complex, irregularly framed structures, or those with large eccentricities between centers of mass and resistance, will require a three-dimensional analysis. Only an outline of some of the elements of modeling will be discussed in this Commentary [Clough and Penzien, 1975; Chopra, 1980; U.S. Army, 1986; Biggs, 1964, SEAOSC, 1977; Wilson and Button, 1982].

There is a series of key assumptions that are common to most analysis models: the structure is assumed to be linearly elastic; small deformation theory applies; structural mass is commonly lumped at a few selected joints or nodes; and energy dissipation (damping) is assumed to be viscous or velocity proportional.

C106.3.1 ***General Stiffness Modeling Considerations.*** Great care must be exercised in developing a model for the structure because a structure is necessarily represented by an idealization containing fewer, conceptually simpler, elements than actually exist in the structure. The engineer must ascertain that the model adequately represents the structure such that the desired behavior and response will be obtained, and that no extraneous or artificial responses occur from use of the simplified model. These artificial responses can result from arithmetic errors introduced by truncation, often caused by large differences in stiffness; the introduction of rigid elements (stiff ends); improper stiffness assignments (particularly where different types of elements intersect at joints); and from not recognizing the fundamental behavior pattern of the structure in preparing the model.

Three-dimensional models are required when the use of a two-dimensional model would obscure some significant information. It should be noted that by themselves, two-dimensional models are unable to reflect orthogonal effects and torsion in the structure (see Appendix 1D to Commentary Chapter 1).

1. **Finite Joint Size.** In modeling beams and/or columns of frame structures, beam elements are usually considered as concentrated about their principal axes and then geometrically represented as lines. Intersections of such lines at model node points are the physical connection points in the real structure. In mathematical models, these physical connection points (called nodes or joints) are normally treated as single point locations that have no size dimension. For relatively slender beam or column elements, the assumption that the joints are dimensionless or of negligible size is satisfactory. However, where beams and columns have significant depth at intersections, the overall structure is stiffer than as represented by the assumption of zero joint size, and this must be considered in the model formulation.
2. **Diaphragm Flexibility.** Diaphragms are generally classified into three groups: rigid, semi-rigid, and flexible. These terms are relative and depend on the stiffness of the lateral force resisting vertical elements. For example, a rigid diaphragm in a frame structure may be semi-rigid in a shear wall structure. The response of a structure with a large plan eccentricity could be very sensitive to the diaphragm flexibility and appropriate evaluations of stiffness should be made to model this element correctly. See Commentary Section C105.6 for discussion of diaphragm rigidity/flexibility.
3. **Section Properties.** The selection of section properties for the structural model is complex and requires considerable judgment to properly understand

and represent the structure's expected performance. Reinforced concrete and masonry are particularly difficult because they are nonhomogeneous materials with properties that can vary significantly at different levels of response. For concrete and masonry, a first approximation would be to use gross uncracked section properties and the code modulus of elasticity. Assuming cracked sections and transformed steel area throughout, or using the moments of inertia specified in the ACI code for deflection calculations, may be an excessive refinement because the modal response results may have to be scaled by Section 106.4.1.3 of this section, and the period is limited by Section 105.2.2.2. However, where deformations are of concern in either meeting the limitation on drift, P-delta, or the $3(R_w/8)$ factored deformation provisions, it is recommended that a representation of cracked section stiffness be used. All significant resisting elements should be included in the stiffness model, such as steel members embedded in concrete or cast-in-place concrete beams and columns that may not be part of the lateral force resisting system. Stiff dummy members should not be used without checking the impact this may have on the results because of potential numerical difficulties.

4. **Boundary Conditions.** Boundary conditions are the constraints in a model designed to match physical conditions at the edges of a structure or component. Boundaries for models required for seismic response are usually those that move with the ground at the location of the specified base motions. Boundary elements or nodes may be utilized in response analyses to model several conditions or effects. Base elements or nodes may be fully or partially constrained to model the support stiffness of a structure, including its foundations. Most commonly, nodes at "ground" are fixed in rotation or translation (or both) in order to model boundary restraint. It is assumed in these Requirements that most structures will be analyzed without explicit consideration of soil stiffness. Where the fixed base assumption produces unconservative results, soil flexibility should be considered in the analysis.

C106.3.2 ***Mass Representation.*** Distributed mass commonly will be either lumped at the nodes in some fashion, or placed as a single mass lumped at each floor level. Where lumped mass is used, as is the case in most computer programs, care must be taken to ensure:

1. Any mass eccentricity of the real structure is properly retained.
2. Application of the resulting forces on the structural model does not cause erroneous beam shortening.

The mass of mechanical equipment, curtain walls, and other nonstructural elements must be considered. Story masses should be lumped *only* when the diaphragm is rigid; otherwise they should be distributed.

There are computer programs that permit an actual mass distribution within individual finite elements. These programs use the consistent mass formulation. Care must be taken to understand the method used in the computer program selected for the analysis.

C106.3.3 ***Damping.*** Energy dissipation is normally idealized as viscous or velocity-proportional damping in order to allow the equations of motion to be uncoupled and solved as a set of linearly independent equations. Each independent equation determines the response for one mode. For convenience, damping is generally stated as a percentage of critical damping or as a ratio of damping to critical damping; hence the term damping ratio. The response spectrum curves given in these Requirements are based on damping that is 5 percent of critical damping; other spectra that may be used should also have this 5 percent damping value. Damping in time-history analysis should be 5 percent, unless a higher value can be substantiated by data.

C106.4 Dynamic Analysis Methods

While two methods of dynamic analysis are presented, it is expected that a majority of engineers will use the response spectrum analysis procedures (see Section 106.4 and Appendix 1D to Commentary Chapter 1). The time-history analysis procedure (Section 106.5) is included for use where it is important to represent inelastic response characteristics or to incorporate time-dependent effects when computing the structure's dynamic response.

C106.4.1 ***Response Spectrum Analysis.*** The response spectrum analysis is the preferred method for most buildings. It is strongly recommended that the engineer first perform an equivalent static analysis to help verify the model and to provide a basis for comparison of the response spectrum results. It is considered good practice to plot the significant mode shapes as a means of both checking the model and gaining insight into how the structure performs. Appendix 1D to Commentary Chapter 1 provides the necessary relations and concepts for application of the minimum requirements.

C106.4.1.1 ***Number of Modes.*** See Appendix 1D to Commentary Chapter 1 for an analytical approach to determining the required number of modes. Demonstration that 90 percent of the participating mass of the structure is included in the calculation of response for each principal horizontal direction is, in SEAOC's judgment, an acceptable procedure.

C106.4.1.2 ***Combining Modes.*** Each required response quantity, such as the peak member forces and displacements, element forces and deformations, or the story shears and drifts shall be evaluated for each significant mode and these individual mode response quantities may be combined to evaluate the total structural response according to the modal combination procedures described in this section of the Commentary. The same method of modal combination should be used for each response quantity. It must be noted that in modal analysis, when the individual modal force quantities are combined, the related structural components are no longer in equilibrium. This result occurs because the individual modal maximums do not all occur at the same time. Equilibrium and the consistency of displacements with forces can be checked only on an individual mode basis [Wilson et al., 1981; Der Kiureghian, 1981; Wilson and Button, 1982; Biggs, 1964; U.S. NRC, 1976]. See also Appendix 1D to Commentary Chapter 1.

Methods such as square-root-sum-of-the-squares (SRSS) and Complete Quadratic Combination (CQC) (see Appendix 1D, Part 3 to Commentary Chapter 1), used to combine the effects of the several modes, are approximate and account for the fact that all modal maximums do not occur at the same time. There is no assurance that either method will produce conservative results; however, under the proper conditions, each method will produce results of acceptable accuracy.

The adequacy of the SRSS method depends on the ratio of the modal periods and the modal damping ratios. This method will be acceptable when the modal damping ratios are no greater than 5 percent, and the ratio of the periods of any higher mode to any lower mode is 0.75 or less. For larger damping ratios or larger modal period ratios (approaching 1.0), modal coupling effects may become important. Such effects are not accounted for in the SRSS method, but are incorporated when the CQC method is used. The CQC method (described in Appendix 1D to Commentary Chapter 1) is a general and theoretically well-based modal combination method that reduces to the SRSS method when the modal periods are well spaced and the modal damping ratios are small. Thus, the CQC method will always be an acceptable modal combination method. The damping ratio employed in the CQC evaluation should correspond to that used for the response spectrum input.

Modal combinations present several important problems in the interpretation of results. First, all computed terms are positive. Second, the value associated with each term may correspond to a different point in time. Thus member and joint equilibrium cannot be checked; moments, shears, and deformations at points between the nodes in the model cannot be directly calculated. The engineer needs to consider these conditions when using the terms, and should assign signs to the individual terms to ensure that the results are conservative. An examination of individual modes may be useful in those assessments. Some code provisions, such as UBC Division IX §2251.H1, require that the engineer know if a member is in single or double curvature, and the predominant mode of response could be used to determine this condition.

C106.4.1.3 ***Scaling of Results.*** The earthquake force levels of subsection 105.2 for static analysis reflect the influence of structural period, ductility and damping (energy dissipation) for various structural systems. Because it is difficult to reflect these influences on a consistent basis for dynamic analysis design, SEAOC included a procedure for scaling the dynamic response. This scaling is used to make dynamic analysis results consistent with the results of the static design approach. The static design approach has gained acceptance since regular buildings designed by this approach have, in general, performed acceptably in earthquakes. Zsutty and Shah [1981] provides a discussion of the relationship between traditional design load levels, structural configuration and details, and building performance. SEAOC has determined that this scaling procedure is appropriate for design, but it is an interim procedure subject to future replacement. Great care should be exercised in interpreting scaled results to ensure that they are physically reasonable and appropriate, since many quantities may not actually scale linearly with base shear.

For example, it is possible that certain characteristics of a particular ground motion and/or building system may lead to either systematically high or low values of dynamic response. Such aspects of building response could be lost when current scaling procedures are employed. Furthermore, scale factors obtained using base shear will not necessarily be the same as scale factors based on maximum drift, member forces, or other quantities of importance for seismic design.

C106.4.1.3.1 This subparagraph requires the base shear resulting from a dynamic analysis be increased to that required by Section 105. All corresponding forces also must comply with this increase. For the purpose of evaluating the base shear that will form the basis for scaling the dynamic analysis results, the period of the first mode may be used in calculating the *C* factor by Section 105.2.2.2 Method B. The value of *C* shall be consistent with the calculated period, but shall not be less than 84 or 80 percent of the value obtained by use of the period from Method A. Consistent with Section 105.8.3, this limitation need not be applied to the scaling of displacements.

Dynamic procedures are considered to provide a somewhat improved, or more accurate, force distribution. Therefore, a base shear of 90 percent of that given by the static force procedure of Section 105 is permitted for a regular structure when a dynamic analysis is performed. Also, when dynamic analysis (Section 106) is used, then the fundamental (first mode) period *T* is provided (Method B). This qualifies for the advantage given in Section 105.2.2.2, where the resulting *C* coefficient can be as low as 84 or 80 percent of the Method A value. For flexible systems, such as frames, it is possible to qualify for both allowed reductions (90 percent and 84 or 80 percent). However, in order to prevent excessively low base shear resulting from combining both reductions, the lower limit of 84 or 80 percent of the Method A value was set.

C106.4.1.3.2 This is a permissive section in that the engineer is permitted to decrease the dynamic base shear to that required by Section 106.4.3.1. However, there may be cases where the engineer or owner may wish to use force levels above the minimums established by these Requirements in order to provide more damage control or continuity of operation. Note that there are special Requirements for S_4 sites (see Section C106.4.1.2).

C106.4.1.4 ***Directional Effects.*** See Commentary Section C105.1 for details. Effects of vertical ground motion on horizontal cantilevers and prestressed elements can be accommodated by using the static design procedures of Section 105.10 or by using a dynamic analysis with a vertical component, as noted in Section 106.2.5. However, the resulting forces may not be less than those required by the static force procedure.

C106.4.1.5 ***Torsion.*** The occurrence of significant torsional motions can lead to increased loads in the building's lateral force resisting system and is therefore an important seismic design consideration. Such torsional motions can be strongly coupled with the building's lateral motions if large eccentricities exist between the centers of story resistance and the centers of floor mass, and if the natural frequencies of the building's normal modes are closely spaced [Der Kiureghian, 1981; Kan and Chopra, 1977]. For such conditions, a three-dimensional model should be used to conduct the dynamic analysis (as discussed in Commentary Section C106.3) and the CQC approach is recommended for combining the maximum modal responses (see Commentary Section C106.4.1.2).

Accidental torsional effects prescribed in Section 105.6 are also required for the dynamic analysis procedures. Torsional motions can occur in a building even if its centers of mass and resistance are coincident and the natural frequencies of its predominant modes of vibration are well spaced. Such torsional motions, termed accidental torsion, can arise from several factors not typically considered in the dynamic analysis of buildings, such as:

1. Spatial variations of horizontal input motions.
2. Rotational components of the ground motions.
3. Effects of nonstructural elements (e.g., partitions, stairs, etc.) on the building's stiffness and inertial characteristics.
4. Actual distribution of dead and live loads.
5. Uncertainties in defining the building's material properties for the dynamic analysis.

To account for such effects, torsional moments due to accidental torsion can be computed using the static procedures given in Section 105.6, and then distributed to the various members of the building's lateral force resisting system to obtain the corresponding member forces. This distribution can be accomplished by either an equivalent static load method or a dynamic analysis method. In the static method, the accidental torsional moments at each level may be calculated as the product of the equivalent static force from Section 105.4 times the 5 percent eccentricity specified in Section 105.5.2. The resulting moments are applied as pure couple loadings, all of the same sense, at their corresponding levels. The effects of these couple loads are then added, as an increase, to the results of the dynamic analysis and scaled according to the Requirements of Sections 105.5.3 and 106.4.1.3.

For the dynamic method and nonflexible diaphragms, it is required to displace the mass in the dynamic model to alternate sides of the calculated center of mass and use a three-dimensional analysis to calculate the effects directly. For highly eccentric structures, care should be taken to ensure the critical location of this mass displacement to obtain the highest load effect. Additional details and Requirements can be found in Section 105.6 and its Commentary.

C106.4.1.6 ***Dual Systems.*** The dual system Requirements presented in this section are, by reference, the same as those described in Section 104.6.4. The determination of base shear for the dual system is for the combined system. Section 104.6.4.2 requires a special moment resisting frame capable of resisting at least 25 percent of the total base shear, in effect providing a "back-up" capacity. There is no theoretical justification to perform a response spectrum analysis on the frame by itself, since it is fully connected to the shear walls or bracing elements. Consequently, permission is given to the engineer to perform a static analysis of the frame to ensure it is capable of carrying the specified base shear.

C106.5 ***Time-History Analysis.*** The time-history procedure described in this section uses a time-dependent representation of the seismic input motions. The procedure is applicable to either a linear elastic or a nonlinear model of the building. It computes the time-dependent dynamic response of the model through numerical integration of the model's equations of motion. Time-history analysis procedures are typically more complex to implement than are response spectrum analysis procedures. This increased complexity and corresponding increased costs may not be warranted for most structures, particularly in view of the uncertainties in modeling and in defining a suitable ensemble of input time histories. Nevertheless, such procedures do have certain advantages over response spectrum methods, particularly for special situations where it is important to represent inelastic behavior or to compute time-dependent response characteristics.

It is the intent of this section that the resulting structure be in general compliance with these Requirements. When the engineer, for whatever reason, uses the time-history analysis procedure, complete backup information must be provided. This is to allow the building official to determine that the proposed structure is in general compliance with the force levels permitted in these Requirements.

It is strongly recommended that both a static force analysis and a response spectrum analysis be performed before any time-history analysis is performed. This provides a means of validating the analysis and gives the engineer something with which to compare the time-history analysis results. If this is not done, results may be questionable and difficult to interpret.

C107 UBC §1630 Lateral Force on Elements of Structures, Nonstructural Components, and Equipment Supported by Structures

Section C107 provides design Requirements for miscellaneous elements of structures, nonstructural components, and equipment supported by structures. Included are requirements for the items themselves, requirements for the means by which such items are attached to the supporting structure, and special requirements for items that could be defined as safety related.

The following definitions are provided here to better distinguish the various categories of items covered by this section. They apply only to the discussions of Section C107.

Elements. Miscellaneous assemblies or portions of structures having a structural function related to the structure and include but are not limited to:

1. Walls, including parapets, exterior walls, and interior bearing and nonbearing walls.
2. Penthouses, except where framed by an extension of the structural frame.
3. Prefabricated structural elements other than walls.

Components. Permanent assemblies not having a structural function and include but are not limited to:

1. Exterior and interior ornamentation and appendages.
2. Chimneys, stacks, and towers.
3. Signs and billboards.
4. Storage racks, cabinets, and book shelves, including contents, over 6 feet tall.
5. Suspended ceilings and light fixtures.
6. Access floor systems.
7. Masonry or concrete fences over 6 feet tall.
8. Partitions.

Equipment. Consists of mechanical, plumbing, and electrical assemblies attached directly, or by means of one or more equipment supports, to the structure and include but are not limited to:

1. Tanks and vessels.
2. Boilers, heat exchangers, fired and unfired pressure vessels, tanks, turbines, chillers, pumps, motors, air-handling units, cooling towers, transformers, and switchgear and control panels.
3. Piping, conduits, ducts, and cable trays.
4. Emergency power supply systems and essential communications equipment.
5. Temporary containers of flammable or hazardous materials.

Some large equipment, such as boilers and turbines, may be constructed within a structure but be isolated from that structure. In such cases the equipment should be considered as a nonbuilding structure (see Section 109).

Special Safety-Related Equipment. Equipment needed after an earthquake, fire, or other emergency, or equipment that contains a sufficient quantity of explosive or toxic substances which, if released, would threaten the life safety of the general public. Special safety-related equipment does not include equipment that threatens life safety solely through structural failure.

Equipment Supports. Those structural members or assemblies of members, that include, but are not limited to, baseplates, frames, skirts, skids, hangers, springs, legs, lugs, or saddles that transmit gravity loads between the equipment and the structure. Equipment supports also include members, such as braces, struts, snubbers, and bumpers, that transmit lateral forces and provide structural stability for the equipment to which they connect.

Attachments. The means by which elements, components, equipment, and/or equipment supports are secured to the seismic resisting system of the structure. Such attachments include, but are not limited to, anchor bolts, welded connections, clamps, clips, cables, and fasteners.

Rigid. Equipment is considered rigid when the equipment, its supports, and its attachments considered as a single dynamic system, has a fundamental mode of vibration with a natural period less than 0.06 seconds (natural frequency greater than 16.7 Hz). A rigid piece of equipment supported on vibration isolation devices or other flexible supports shall be considered flexible. For complex equipment, evaluation of the vibratory characteristics may be difficult. However, if the equipment can be reasonably modeled or tested, the fundamental mode of vibration may be characterized as that mode that has the greatest mass participation.

C107.1 General

This section applies to items such as elements, components, and equipment attached to structures whose dynamic behavior does not substantially alter the structure's response. The design lateral force coefficients for items attached to structures are generally higher than the force coefficients used for the structure's design. There are four basic reasons to provide higher lateral force coefficients for attached items:

1. Absolute accelerations acting on items supported by the structure above ground level are generally greater than at ground level.
2. Additional amplified response can occur within items unless they themselves are quite rigid.
3. The items themselves, except when manufactured from and supported by ductile materials, may lack the redundancy or energy absorption properties that allow the rational reduction of force levels used for design of the items.
4. Attachment failures should be minimized.

Section 107 is focused on the design of attachments that transfer from a supported item to the seismic resisting system of the structure. Diaphragms are now considered as part of the structural system and are no longer covered by this section (see Section 108.2.9). The design of exterior elements also includes deformation Requirements as outlined in Section 108.2.4. The design of nonbuilding structures is now considered in Section 109.

C107.1.1 ***Friction.*** Usually friction resulting from gravity loads should not be used to resist seismic forces. However, friction achieved by clamping and friction caused by seismic overturning forces may be used. Friction clips and beam clamps have historically performed poorly in earthquakes. Where friction provided by mechanical clamping (e.g., a beam clamp) is used to resist lateral loads, the engineer should verify that an adequate load transfer mechanism exists. Eccentricities in the load path, prying forces, etc., must not be overlooked. If friction caused by seismic overturning forces is used, the structural system must have sufficient strength and rigidity to transmit the overturning forces, and the supports and foundations (especially edge portions) in the friction load path must be designed to resist the apportioned seismic shear transferred by friction.

It is not the intent of Section 107 to prohibit unrestrained components and equipment. Unrestrained components and equipment may be permitted, particularly temporary and moveable items, such as heavy furnishings, provided that the safety of occupants and the public will not be compromised. Items with low ratios of height-to-base width or length that are inherently stable against overturning may be unrestrained. In such cases, lateral movement may be limited by friction forces based on gravity forces with reductions due to vertical earthquake accelerations, as considered appropriate by the design engineer. The maximum lateral movement of components and equipment that may occur under maximum expected ground motions should be estimated and incorporated into the design of utility connections, seismic restraints, and space shall be provided to accommodate component or equipment movement.

C107.1.2 ***Equipment Design.*** This provision requires that for some equipment, not only the attachments, but also the equipment itself must be designed to resist seismic forces. This provision states that nonrigid equipment, for which "structural failure of the lateral force resisting system. . . would cause a life hazard," must also be designed to resist lateral forces. Although the goal of all seismic design is to minimize hazards to life, these Requirements intend that a special category of concern be defined where the perceived hazard is greater than normal. Several complex interrelated considerations are raised by this new requirement that seismic design must sometimes extend to the equipment itself, and not be limited to attachments. These considerations, which include equipment function, scope of covered equipment, rugged equipment, and responsibility, are discussed in Section C107.1.3 through C107.1.8, below.

C107.1.3 ***Equipment Support Design.*** It is intended that Section 107 address equipment supports that transfer seismic forces from the equipment through attachments to the seismic resisting system of the structure. Equipment supports, even when supplied by the manufacturer, should be designed in accordance with these Requirements. Equipment supports may include equipment appurtenances that are compact or integrally connected to and manufactured with the equipment, and may transfer seismic as well as weight and functional forces to the structure. When reviewed however, these equipment appurtenances should be of sufficient strength to adequately transfer the combined seismic, weight, and functional forces to the equipment supports and attachments, as appropriate.

C107.1.4 ***Equipment Function.*** One consideration, necessitated by Section 107.1.2, is whether equipment function is to be addressed. There are two situations where equipment "structural" failure or loss of "function" could pose a hazard.

One special category is where loss of structural integrity causes a loss in physical connectivity or restraint under seismic motions, resulting in a direct life hazard. An example is where the entire item or a part of the item breaks off and falls, topples, slides, or otherwise moves, posing a threat to occupants, or blocks a means of egress or an exitway. In these cases, the use of bumpers, braces, guys, or gapped restraints may protect the occupant, even if the item itself is damaged.

The second situation is where the failure of equipment to perform a required function could cause a more indirect life hazard. Such equipment could be defined as "special safety-related." Examples include fire protection piping or a standby power system in a hospital. Another example is a vessel or piping that contains sufficient quantities of highly toxic or explosive substances such that a release would be hazardous to building occupants or the general public. Emergency equipment should be located where there is the least likelihood of damage due to an earthquake. Such equipment should be located at or below ground level, and where it can be easily maintained to ensure its operation during an emergency. Judgment may also suggest a more conservative design than would otherwise be required by these Requirements, related in some manner to the perceived hazard. A list of substances considered acutely hazardous (highly toxic or explosive) by the State of California is provided in Appendix C of *Guidance for the Preparation of a Risk Management and Prevention Program* [OES, 1989].

It is intended that the Requirements of Section 107 apply to the design of both of the above categories of equipment. However, these Requirements only explicitly address structural integrity (connectivity and containment of contents) and stability. Equipment function (operability), although obviously important, is not specifically addressed. Although function is not explicitly addressed, it may be concluded on the basis of past earthquake experience that if structural integrity (connectivity and containment) and stability are maintained, function (operability) after an earthquake is probable for most types of equipment, although by no means assured. Also, these Requirements are not intended to ensure that mechanical joints in equipment providing containment, such as flanged joints, will remain leak-tight in an earthquake. It is assumed that future editions of these Requirements will include provisions that more directly address function and operation of safety-related equipment.

C107.1.5 ***Scope of Covered Equipment.*** It is not intended that all equipment, or that all parts of equipment, be designed for seismic forces. For example, it is not the intent of these Requirements to require the seismic design of shafts, buckets, cranks, pistons, plungers, impellers, rotors, stators, bearings, switches, gears, nonpressure retaining casings and castings, or similar items. Determination of whether the Requirements of Section 107 should be applied to the seismic design of a specific piece of nonrigid equipment will sometimes be complex, and even unclear.

Judgment will be required if the intent of these Requirements is to be fulfilled. The following discussion may serve as guidance.

Since the threat to life is the primary consideration in seismic design, it should be clear that a nonessential air handler package unit that is bolted to a mechanical room floor and stands less than 3 feet above the floor is not a threat to life safety as long as adequate anchorage is provided. Therefore, the air handler itself need not be designed for seismic forces. Only the attachments (anchor bolts) need be designed to restrain the movement of the air handler during an earthquake so as to not harm any occupant of the mechanical room. On the other hand, a 10-foot-tall tank on 6-foot-high angle legs (braced or unbraced) mounted on the roof or near a building exitway does pose a hazard. It is the intent of these Requirements that the tank legs, the connections between the roof and the legs, and the connections between the legs and the tank be designed to resist seismic forces. Alternately, restraint of the tank by guys or additional bracing could be acceptable. If the tank contained a highly toxic substance, the tank itself would likely require seismic design.

C107.1.6 ***Rugged Equipment.*** Some equipment consists of complex assemblies of mechanical and electrical parts that typically are manufactured in an industrial process that produces similar or identical items. Such equipment may include manufacturer's catalog items and is often designed by empirical (trial and error) means for functional and transportation loads. A characteristic of this equipment is that it may be inherently rugged. Rugged, as used herein, refers to an ampleness of construction that gives such equipment the ability to survive strong motions without significant loss of function. By examining such equipment, an experienced design professional should usually be able to confirm such ruggedness, as well as assess whether the equipment is rigid or not. The assessment of the ruggedness and rigidity of the equipment will then determine the need for and the appropriate method and extent of the seismic design or qualification efforts. The discussion in Section C107.1.7 regarding design attributes of certain equipment may provide guidance in this assessment.

From the above, it may be concluded that it is not generally intended that these Requirements be applied to the design (other than for anchorage) of most air handlers; compressors; pumps; motors; engines; generators; valves; pneumatic, hydraulic or motor operators; fans; chillers; small horizontal vessels or heat exchangers; evaporators; heaters; condensers; motor control centers; low or medium voltage switchgear; transformers with anchored internal coils; small factory manufactured boilers; inverters; batteries; battery chargers; and distribution panels. In general, such equipment has been found to be rugged.

These Requirements are intended to require the design of nonrigid special safety-related equipment, whether the equipment is placed in Occupancy Category I (Essential), Category II (Hazardous), Category III (Special), or Category IV (Standard) structures. For example, whether or not a structure is Occupancy Category II (Hazardous), a tank or vessel and associated transfer piping that contains highly toxic or explosive materials must be designed to resist seismic forces.

Conversely, not all equipment in an Occupancy Category II (Hazardous) structure should be required to meet seismic requirement commensurate with the structure's classification.

C107.1.7 ***Equipment Lists According to Design Attributes.*** Equipment covered by Section 107 may be divided into two general groups. One group consisting of assemblies of simpler shapes and forms for which national codes and standards often exist for their analysis or qualification for seismic motions. The second group consists of complex mechanical and electrical assemblies whose primary function is a mechanical or electrical process for which national codes and standards may not exist. In the design and analysis of both groups of equipment, the concepts of rigidity and ruggedness can often assist the seismic designer of equipment in determining the necessity for analysis and, given that analysis is necessary, the extent and methods by which seismic adequacy may be determined. This section categorizes many different types of equipment to assist the seismic designer of equipment in assessing the seismic attributes of equipment. These lists are by no means complete. The designer should not be too literal in the assessment of equipment attributes. For example, chillers are listed below as both rigid and rugged. But, one chiller may be rigid, while another may not. Both chillers may be rugged and can be reasonably expected to survive earthquake motions without loss of integrity. However, the attachments for the nonrigid chiller should be designed to withstand the seismic amplification of the equipment. Thus, these lists should be viewed as qualitative only.

The first group is equipment that may usually and reasonably be analyzed for seismic forces. Also, this class of equipment would often be designed in accordance with established national codes or standards, which may have acceptance criteria for seismic loads. This equipment may include, but not be limited to, those items listed in Equipment Lists C107-1 through C107-3. Also note that several of the items listed in Equipment Lists C107-1 and C107-2 would normally be designed in accordance with Section 109.

The second group of equipment consists of complex assemblies of mechanical and electrical parts, including manufacturer's catalog items. This class of equipment may not be easily designed for seismic forces and motions, but observed performance of similar equipment during seismic events may provide an adequate level of confidence for the experienced design professional to reasonably predict the seismic performance of the equipment being considered. This equipment may include, but not be limited to, those items in Equipment Lists C107-4 and C107-5.

It may be necessary to design for higher seismic loads than provided for in these Requirements to ensure that special safety-related equipment maintains structural integrity or can function following a seismic event.

Equipment List C107-1. *Equipment that may in part or as a whole be subject to seismic amplification*

- Chimneys and stacks
- Large field-erected boilers
- Tall flat bottom storage tanks
- Tall bins and silos
- Tall vertical pressure vessels
- Guyed and freestanding towers
- High voltage switchyard equipment

Equipment List C107-2. *Equipment that may not be subject to seismic amplification because it is rigid*

- Heat exchangers
- Heaters
- Evaporators
- Condensers
- Short vertical pressure vessels
- Broad flat bottom storage tanks
- Horizontal pressure vessels
- Short bins and silos

Equipment List C107-3. *Equipment in which the potential for seismic amplification is reduced by ductile behavior (when fabricated from steel and steel alloys, copper and copper alloys, and aluminum)*

- Piping
- Tubing
- Plumbing
- HVAC and process ducts
- Electrical conduit
- Electrical cable trays

Note: Piping and cable trays which are supported on the ground using flexible cantilever supports have a history of performing poorly during seismic events; amplified forces should be considered in the design of these supports.

Equipment List C107-4. *Complex equipment that may typically be considered both rigid and rugged*

- Valves
- Engines
- Motors
- Generators
- Turbines
- Batteries
- Chillers
- Inverters
- Horizontal pumps
- Pneumatic operators
- Hydraulic operators
- Motor operators
- Air and gas compressors
- Small factory manufactured boilers
- Transformers with anchored internal coils

Note: Some complex equipment, such as valves and valve operators, turbines and generators, pumps and motors, may be functionally connected by mechanical links not capable of transferring the seismic loads or accommodating the seismic displacements, and may require special design considerations such as a common rigid support or skid.

Equipment List C107- 5. *Complex equipment that may typically be considered rugged, but not necessarily rigid*

- Vertical pumps
- Distribution panels
- Battery chargers
- Motor control centers
- Air handling equipment and fans
- Low and medium voltage switchgear
- Instrumentation cabinets

Note: Some equipment, which by itself may be considered rugged, may nonetheless be vulnerable to seismic amplification because it is supported by flexible legs, frames, racks, or other assemblies of structural shapes.

C107.1.8 ***Responsibility.*** Responsibility is another consideration necessitated by the Requirement of Section 107.1.2 that some equipment be designed to resist seismic forces. These Requirements provide forces to be used in the design of equipment and equipment attachments. They do not specify, and they make no judgment as to which design professional is responsible for the seismic design (or review) of any specific piece of equipment or its attachments. For equipment installed at the time of construction, responsibility could be that of the structural engineer who designs the structure, a mechanical engineer employed by the manufacturer who fabricates the equipment, another engineer retained by the mechanical contractor, or perhaps still another design professional. Historically, design of equipment has usually not been the responsibility of the structural engineer responsible for the design of the structure. It is not the intent of these Requirements to change any current contractual practices.

Typically, on new construction, the lead design professional on a project will negotiate or otherwise determine, in conjunction with the design professional responsible for the functional design of the equipment, how such seismic design of covered equipment will be performed, which design professional will have responsibility for its performance, and whether on a contractual or fee basis. These Requirements do not deal with these issues.

For equipment installed at some time after completion of construction, the original project team may no longer be available. Thus, the owner, manufacturer, or mechanical contractor may need to obtain assistance in determining the need to apply these Requirements.

Recent postearthquake surveys have revealed poor quality control with respect to attaching equipment to structures. It is important that a design professional or a qualified inspector check that the attachments are provided in general conformance with the design documents.

Section 107.1 exempts the attachments for floor- or roof-mounted equipment weighing less than 400 pounds (181 kg) from design (note that Section 107.1 does not say that the equipment does not need to be designed). An item of special safety-related equipment should not be exempt from being seismically designed or restrained, or conforming to the other Requirements of Section 107 just because its weight is less than 400 pounds. Also, although it is not specifically required that equipment and furniture weighing less than 400 pounds be attached to the structure, an evaluation of the inherent fragility or ruggedness of equipment or furniture, or the hazard created if the item slides, topples, or otherwise moves as a result of earthquake effects, may require the design of seismic restraints.

Many equipment items are designed in accordance with national codes and standards. For example, pressure vessels and pressure piping are typically designed in accordance with codes and standards sponsored by the American Society of Mechanical Engineers and approved by the American National Standards Institute. Many national codes and standards have allowable design stresses and other acceptance criteria for the variety of operating and environmental loads, including earthquake loads, to which the equipment item may be subjected. However, these same codes and standards may or may not contain seismic force requirements. It is intended that these Requirements be used to develop seismic forces to be accommodated by the allowable stress and other acceptance criteria of appropriate national codes and standards approved by the design professional responsible for the functional design of the equipment.

C107.2 Total Design Seismic Force

C107.2.1 The values of the coefficients used to determine F_p are similar to values of previous Requirements in that they are judgmental. They were set considering the results of analyses based on linear elastic response of multi-story buildings with generally regular configurations. These analyses indicate that for most buildings:

1. Peak accelerations of the upper stories can be three or more times the peak ground accelerations.
2. Peak measured accelerations (as a percentage of the acceleration due to gravity) of the upper stories are generally between 1.6 to 2.3 times the peak base shear coefficient (the ratio of the elastic base shear to the weight of the building).
3. Peak acceleration for the upper half to two-thirds of a building are nearly constant, except at the roof, where they are significantly higher). These peaks do not occur at the same time for each story level, but occur at different times since the modes of vibration combine differently for each story. In both the 1989 Loma Prieta and 1994 Northridge earthquakes, peak floor accelerations well over 1g were recorded [Liu et al., 1977; CDMG, 1984].

The C_p factor for design was first implemented in a 1933 ordinance in Los Angeles, probably as a direct result of studies begun in 1928 by the California State Chamber of Commerce and later adopted as part of the 1937. These ordinances included provision for the design of walls, parapets, appendages, towers, and tanks, that have carried through to the present time. Minor changes were made with the adoption of the first SEAOC Blue Book in 1959. The 1974 Blue Book retained the same C_p values as previous editions, although the C value for the design of the structure was increased. "S" and "I" coefficients were included to be consistent with the base shear formula then in use. Studies by SEAOC indicated that S does not have the same importance for the design of elements as it does for buildings. Therefore, the "S" coefficient was eliminated in 1980. In the future, "S" may be reintroduced into Equation 107-1 if further study indicates that such a term is necessary.

An "I_p" coefficient has been introduced to permit designation of a seismic importance factor for the nonstructural item. In the past, the seismic importance factor was the same for nonstructural items as for the structure. The "I" factor was originally 1.5 for structures in Occupancy Categories I and II (Essential and Hazardous), but this was subsequently reduced to 1.25. However, an I_p value of 1.5 is considered necessary for the design of most nonstructural items in Occupancy Category I and II facilities, as well as in the design of special safety-related equipment in all facilities and occupancies.

The values of C_p listed in Table 104-7 are for all elements, components, and equipment. The flexibility of elements and components need not be considered in design, and need not be checked for possible dynamic amplification effects ($C_f = 1.0$).

Definitions of "rigid" and "flexible" are provided at the beginning of this Section C107 of the Commentary. The definition of rigid (having a natural period of less than 0.06 seconds) was selected judgmentally. Below that period, the energy content of seismic ground motion is generally held to be very low, and therefore the structure response is not likely to be excessively amplified. Also, it is very unlikely that any building will have a first mode period as low as 0.06 seconds, and it is even unusual for a second mode period to be that low. Thus, the likelihood of either resonant behavior or excessive amplification becomes quite small for equipment having periods below 0.06 seconds.

The revised Equation 107-1 gives the horizontal seismic design force for an item in a form similar to the base shear formula of Section 105. "Z" is the Zone (acceleration) factor, 0.4 in Zone 4, which applies to much of California. Since the product of C_f and C_p is less than or equal to 2.0, then the maximum value for $ZC_p C_f$ is 0.8. In those cases where Z is different from the values in Table 104-1, as discussed in Section 104.2, the value used for design should be the same as used for the structure in order to maintain consistent performance. The engineer may choose to use higher design values for economically or operationally important elements or equipment.

The exception for mechanical piping and ducting and electrical conduit, including cable trays manufactured from ductile materials is provided because recent testing and considerable experience suggests that these items appear to be capable of considerable inelastic strains prior to failure (see Equipment List C107-3). In recognition of this ductile capacity, the C_p coefficient from Table 104-7 for piping, ducting, and conduit, even if such items are flexible, is judged adequate and need not be increased as may be required for other flexible equipment. This Requirement provides a factor similar to the system quality factor, R_w, for the structure. The exemption is also based on the presumption that ductile piping, ducting, and conduits are likewise supported and attached to the structure with ductile materials. It is envisioned that for other flexible equipment where ductile capacity and inelastic behavior without failure can be substantiated, that a similar direct use of the values of C_p in Table 104-7 without amplification may also be justified.

The $C_p=2.0$ value for unbraced cantilever elements, ornamental, and other appendages is the maximum value required by these Requirements. The conservative value is specified, since such elements lack recovery capability after yield of the member at the support and would therefore create a great hazard to the public in the event of their failure. The present value of $C_p=0.75$ for all other items is consistent with the level of design used for the structure. This latter value is based on consideration that these items are able to tolerate more damage without the progressive brittle failure typical of $C_p=2.0$ elements. For the purpose of computing anchorage forces to a diaphragm, the C_p for unbraced (cantilever) parapets may be taken as for the braced portion of the wall element.

C107.2.2 No Commentary provided.

C107.2.3 Attachments for flexible equipment (and, as discussed in Commentary Section C107.1.2, sometimes the equipment itself) must be designed for forces that are developed considering possible amplification of floor motion. Ideally, the design force for flexible equipment should be determined by rational methods that consider the dynamic characteristics of the flexible equipment, the support elements, and the structure. However, a rational determination is often difficult and frequently unwarranted. Therefore, a judgmental upper limit of amplification of twice the force on rigid equipment is accepted in lieu of analysis. The flexibility coefficient C_f was added to Equation 107-1 for calculation of the total lateral design force F_p as an explicit reminder to the design engineer. Admittedly, the theoretical dynamic amplification of equipment resonant with structure response could be much higher. However, experience in past earthquakes, mainly the lack of extensive serious damage to anchored equipment for which theory would have predicted such damage, led to the decision to limit the dynamic amplification factor required for design.

C107.2.4 In past editions of these Requirements, the C_p coefficient for items supported at or below grade could be reduced to two-thirds the values shown in Table 104.7 because these items are not subjected to the amplified response found higher in the structure. In the current Requirements, a coefficient C_g has been added to Equation 107-1 to achieve the same end in a more explicit form. C_g has a value of two-thirds for items supported at or below grade, and a value of 1.0 for items supported on a structure. It is envisioned that in the future this coefficient may be used to include amplifications over the height of the structure. To avoid inconsistent requirements for such items, the resulting design lateral forces should not be less than those which would be obtained by treating the item as an independent structure and using the Requirements of Section 109. For ground-supported walls or other components that are also supported laterally above grade at the upper floors or roof level, $C_p = 0.75$, and may not be reduced by the two-thirds reduction factor allowed for laterally self-supported items and components. The reduction is not allowed because of the amplification effects imposed on the items at the upper floors or roof level.

In these Requirements, only horizontal earthquake directions are required to be considered. Vertical ground motion effects are adequately covered by gravity load design.

C107.3 Specifying Lateral Forces

As explained in previous paragraphs, the design of equipment is usually not controlled by the engineer responsible for the structural design of the structure. To ensure understanding of the required design basis, the specifications for equipment should either specify design lateral forces prescribed by these Requirements, identify and reference these Requirements, or identify and reference other Requirements to be used in lieu of these Requirements.

C107.4 Relative Motion of Equipment Attachments

A significant failure mechanism for items connected by means of multiple attachment points to one or more structures is differential seismic motion of the attachment points. Thus, while current Requirements consider drift only for nonductile elements and special safety-related equipment in all seismic zones, nonductile elements and other important equipment in the higher seismic zones should be evaluated for the effects of any significant differential seismic attachment point motions. Differential seismic attachment point motions can occur when equipment is connected to separate foundations; separate structures; separate substructures or floors within a structure; and on equipment, such as piping entering the structure from the soil beneath or beside the structure. Values for differential seismic attachment point motions may be provided or coordinated by the design professional having overall design responsibility for the structure.

C107.5 UBC §1630.5 Alternative Designs

For some types of equipment, nonstructural components and elements of structures, there are national standards that are used to provide a basis for earthquake design or physical qualification. These Requirements do not preclude the use of such national standards, as long as the lateral force, F_p, and overturning moment are not less than 80 percent of the values that would be obtained using these Requirements. The concern expressed by these Requirements is that some standards, based on insufficient seismic experience, will not provide adequate protection from seismic hazards. However, it is not the intent to deny legitimate research or responsible national standards based on that research.

Examples of national standards for design and physical qualification include, but are not limited to:

1. American Water Works Association, Standard D100 for Welded Steel Tanks for Water Storage, ANSI/AWWA D100-84 and D100a-89.
2. American Petroleum Institute, Welded Steel Tanks for Oil Storage, API Standard 650, 9th Edition, 1993.
3. American Concrete Institute, Recommended Practice for the Design and Construction of Concrete Bins, Silos and Bunkers for Storage of Granular Materials, ACI-313R-1977 (Revised 1983).
4. Sheet Metal & Air Conditioning Contractors' National Association, Guidelines for Seismic Restraints of Mechanical and Plumbing Systems, 1992.
5. Institute of Electrical and Electronics Engineers, IEEE Recommended Practices for Seismic Qualification of Class 1E Equipment for Nuclear Power Generating Stations, IEEE Std. 344- 1987.
6. National Fire Protection Association, NFPA 13, Installation of Sprinkler Systems, 1994.
7. American Society of Mechanical Engineers, ASME A17.1, Safety Code for Elevators and Escalators, 1993.
8. American Society of Mechanical Engineers, ASME B31, Code for Pressure Piping, including addenda through 1993.

C107.6 Additional Comments

The seismic demands imposed on equipment are a function of the ground shaking, the supporting structure, the equipment attachment, and the equipment. Determination of these demands is complex and requires a dialogue between equipment designer/manufacturer and structural engineer. In the nuclear power industry, seismic demands on equipment are determined by a complex dynamic analysis of the structure that results in "floor" spectra. These "floor" spectra then become part of the procedure to develop equipment in typical structures. Hence, a more simplified procedure is used herein where the demand is determined more or less independently of the structural characteristics. This simplified method is more easily understood and implemented by design professionals responsible for equipment design, e.g., mechanical and electrical engineers and architects.

This Commentary should be referenced in any procurement specifications. Use of common definitions as set forth herein aid the dialogue between specifier/procurer and designer.

C108 UBC §1631 Detailed Systems Design Requirements

C108.1 General

Detailing and material requirements are at least as important as those for force and displacement. The design force formulas *assume* that the Requirements of this section are satisfied; failure to follow these Requirements invalidates the force formulas. Failure to follow all the provisions of these Requirements for any element can result in a brittle link in the structure and thereby compromise the stability of the entire structure, possibly resulting in collapse or excessive damage.

C108.1.1 ***Combined Vertical and Horizontal Forces.*** Floor live loads and snow loads must be assumed to act concurrently with the seismic forces specified, since they are likely to be acting on members during the earthquake. Roof live loads and design wind loads are not expected to be significant during the expected earthquake, and they are not considered to act concurrently with the seismic loads.

C108.1.2 ***Uplift Effects.*** Seismic loads are cyclic in nature, producing both tension and compressive loads in elements as they resist overturning forces. Consequently, the designer must consider both. There is some uncertainty as to what degree dead loads participate in resisting uplift; therefore, design for uplift requires that only 85 percent of these vertical loads be considered. For strength design of reinforced concrete, this Requirement is satisfied by the second load combination given in UBC §1921.2.7. The 0.85 factor given in this Section 108.1.2 should not be applied to the already factored dead load combinations given in Sections 105.7.2 and 404.1.

As discussed in Commentary Section C105.8.2, the seismic forces in the inelastic structure due to major earthquake response are represented by the factor of $3(R_w/8)$

times specified forces. Therefore, while uplift may not be indicated by the load combination ($0.85D \pm E$) in this Section 108.1.2, it could occur due to the real earthquake loading effects. If the consequences of this possible uplift would be detrimental to the design objectives, the engineer should provide for tensile hold-down resistance for lateral forces greater than those specified.

C108.1.3 ***Orthogonal Effects.*** These Requirements are intended to cover those loading conditions where different directional loadings could overstress members of the lateral force resisting system. This can occur where there is torsional irregularity, a nonparallel structural system, or where a given member (usually a corner column) is a part of intersecting lateral force resisting systems. Of special concern are structures with plan irregularities that can cause the structure to deform in a direction perpendicular to the direction of input ground motion, and structures that do not have orthogonal lateral force resisting systems such as triangular shaped portions of buildings. The word orthogonal is used, since any horizontal loading can be represented by two orthogonal components, one of which can be chosen to be parallel to the principal axis of the structure.

C108.1.3.1 The exception recognizes that the error resulting from ignoring the orthogonal effects is acceptably small if the axial load in the column due to seismic forces is small; using the Requirements in Section 108.1.3.2, 0.20 x F_ax.3 = 0.06 x F_a corresponds to the maximum error that could occur. For strength design of reinforced concrete, this exception may be applied when the load-factored axial load due to seismic forces is less than 20 percent of the column axial strength.

While specific Requirements have been provided for columns, the designer should also consider orthogonal effects when critical members other than columns are sensitive to orthogonal loadings.

C108.1.3.2 Two methods for combining orthogonal loadings are provided. The use of 100 percent of the member forces due to loads applied in one direction and 30 percent of the member forces due to loads applied in the orthogonal direction is most likely to be used. It should be noted that the terms can be positive and negative at different times.

The use of the SRSS methodology for combining orthogonal load effects is most likely to be implemented by computer. Here the squared values of effects in each of the two orthogonal directions are added and the square root of the sum is obtained. Note that for the case of modal response spectrum analysis, the effects from each orthogonal direction are the SRSS (or CQC) values of the individual modal response values. Because the SRSS methodology loses the signs of the terms, use of its results require great care, since knowledge about member equilibrium has been suppressed. This methodology results in only one load case for designing members as opposed to two or four cases required by the 100 percent/30 percent method, above. There is no additional penalty when a response spectrum analysis is used since it also results in the loss of sign information.

Both approaches for considering orthogonal effects are approximations. They were developed with consideration of results for a square building.

C108.2 Structural Framing Systems

C108.2.1 No Commentary provided.

C108.2.2 ***Detailing Requirements for Combinations of Systems.*** The more restrictive detailing ensures that the component has the energy absorption capability required for the system having the highest applicable value for R_w. Since the design requirements set upper limits on R_w, a system can be designed with a few elements that have more capacity than others. Care must be exercised to ensure that elements of individual lateral force resisting systems do not become incompatible by adopting smaller R_w's for a preponderance of elements. For example, if the end members of a two-bay frame have lower R_w values due to their participation in other systems, then the other elements of the frame should be designed to the lower R_w value to preserve acceptable behavior of the frame system.

C108.2.3 ***Connections.*** In some geographical regions, particularly those with low seismicity a practice has evolved that leaves the design and detailing of connections to the fabricator or constructor (notwithstanding its criticism by the American Society of Civil Engineers). This practice is absolutely prohibited by this provision. Connection failures have been a common cause of severe damage and collapse in structures during past earthquakes. It is essential that the engineer design the connections and present the detailed requirements on the construction documents. Also, it is recommended that there be a thorough detailed review of the related connection details of manufactured elements when these are used within the structural system.

C108.2.4 ***Deformation Compatibility.*** Commentary Section C105.2.1 states that the structure's actual deformation may be several times the displacements estimated from elastic action of the forces prescribed in these Requirements. Actual deformations are judged by SEAOC to be approximately $3(R_w/8)$ times the elastic displacements determined from prescribed seismic lateral forces. This factor may be unconservative in some circumstances (for example, very flexible frames with few internal partitions, or at locations of structural irregularity where concentrations of inelastic behavior may occur) and should be increased when the engineer has reason to believe the particular building warrants a higher value. The intent is to provide an added degree of safety for nonspecially detailed members of the framing system to ensure that they can also withstand the anticipated deformation without losing vertical load-carrying capacity.

Deformation compatibility provisions have been largely ignored by the design community. In the Northridge earthquake, deformation-induced damage to elements considered not part of the design lateral force resisting system resulted in structural collapse. Damage to elements of the lateral framing system, whose behavior was affected by adjoining rigid elements, was also observed. This has demonstrated a need for stronger and clearer provisions.

These Requirements emphasize the need for specific design and detailing of elements not part of the lateral force resisting system to accommodate expected seismic deformations. Ideally, deformation compatibility would be dealt with by improving design and detailing requirements on a material by material basis so that deformation ductility is improved and an unrealistic calculation burden is not placed on the design engineer. New UBC provisions proposed by ACI and PCA take this approach for concrete systems. However, since such provisions are not in place for all materials, the above changes are recommended.

The minimum deformation Requirement is intended to address concerns that deformations in shear wall type structures are often significantly underestimated, largely due to neglect of foundation softness and realistic element properties in the cracked state. More guidance is offered on computation of deformations, including the Requirement that "cracked section" stiffness values be used. The 50 percent of gross Section value is admittedly conservative, but such conservatism was deemed appropriate by the SEAOC Seismology Committee until further research is performed. The designer can replace this value with a "rational" analysis. Foundation flexibility and diaphragm deformation caused by cracking and inelastic behavior should also be considered.

Beams and columns not part of the lateral force resisting system must be checked to confirm that they are capable of resisting the stresses or strains induced by the magnified displacement, including the effects of service vertical loads. For steel members, it is usually sufficient to determine that instability will not occur due to buckling modes. The possibility of tensile rupture at points of stress concentrations, such as partial penetration welds, must also be investigated. For reinforced concrete members, compressive strains in columns or beams without special transverse reinforcement of stirrup ties should not exceed 0.003 inches per inch, and the shear strength of the members should be greater than that resulting from moments induced at the ends of the members by the displacements. If the induced moments resulting from the magnified displacements exceed the moment capacity of the elements, then inelastic behavior is required to accommodate the loads and appropriate special detailing must be provided.

The $3(R_w/8)$ factor is based on the $3/K$ value used in deformation compatibility section of the previous editions of these Requirements (see the related discussion in Commentary Section 105.2.1). While it might be considered good practice to keep computed inelastic drift values within the $3(R_w/8)$ times design drift value, the $3(R_w/8)$ factored value is not intended to be a limit on the inelastic drift due to the maximum expected ground motion. If, for example, a set of representative inelastic time-history analyses indicated drift values larger than the $3(R_w/8)$ factored values, then these would be acceptable as long as the engineer could demonstrate that the structure would maintain stability and provide for life safety at the computed levels of inelastic deformation.

The $3(R_w/8)$ factor (and the previous $3/K$ factor) is judged to provide a sufficient measure of inelastic displacement, given that the vertical load-bearing space frame is made up of true beam and column elements; and where in Section C104.6.2 of this Commentary (as in the previous Commentary) it is strongly recommended that nominal moment resistance be incorporated with details and reinforcing according to good design practice. With this true frame and details, it is assumed that if the $3(R_w/8)$ factored displacements can be accommodated without exceeding the strength of the elements, then there will be sufficient inelastic deformation capability to tolerate any possible excess inelastic displacements beyond the $3(R_w/8)$ factored value. It is most important to recognize that this assumption may not apply for other types of vertical load bearing frames without true beam or girder elements. For example, reinforced concrete or prestressed concrete flat plate systems with "beam" elements provided by the slab column strips would require special design and/or displacement control considerations. In these systems, the combination of shear and flexure at the "beam" column joint could result in brittle failure characteristics in the event of excess displacements (see also Appendix 1E to Commentary Chapter 1).

C108.2.4.1 ***Adjoining Rigid Elements.*** This is a very important provision that should be applied, not only to the frame elements that are not part of the designated lateral force resisting system, but to all structural elements that support gravity loads. The adjoining rigid elements, such as stairways, sloping ramps, infill walls both at full and partial story height, can restrict the flexibility of the structural element such that the imposed seismic deformations can cause actions beyond the strength of the element. "Captive" or short column failures are an example that has commonly occurred in past earthquakes. The actual, as-built constraints or end conditions should also be represented. For example, the original analysis may have assumed pinned-end conditions for columns, where the actual slab and/or beam framing details may create nearly fixed-end conditions.

It is also most essential to consider the possible added deformation effects due to flexible or deformable diaphragms. Assumed rigid diaphragm conditions may not be valid for large or irregular diaphragms responding in the inelastic range.

C108.2.4.2 ***Exterior Elements.*** The listed Requirements 1 through 6 are to ensure that the connections do not fail and prevent the element from carrying the applied loads.

C108.2.5 ***Ties and Continuity.*** Requirements are given to ensure that all parts of the structure act as a unit without localized separations, loss of support, or collapse.

C108.2.6 ***Collector Elements.*** The collector (also called drag strut or drag element) includes the connections between the floor or roof diaphragms and the vertical shear resisting elements. It is important that these joints or connections be designed to prevent localized slip failure (or rupture), which would in turn prevent inelastic energy dissipation from developing in the lateral force resisting system as assumed. Particular attention to this problem is required for post-tensioned slab connections to shear walls, and for precast concrete floor and/or wall panel connections.

It is recommended that collector elements and their connections be designed to have the strength to resist $3(R_w/8)$ times the specified seismic design forces. The purpose is to ensure that inelastic energy dissipation occurs in the ductile lateral force resisting elements (frames, braces, walls) rather than in the collectors and connections.

C108.2.7 ***Concrete Frames.*** Previous editions of these Requirements required that concrete frames in regions of strong seismicity have special details to resist lateral loads. This Requirement has certainly been justified by damage to concrete frames experienced in the 1967 Caracas, Venezuela, 1971 San Fernando, 1985 Mexico City, and other subsequent earthquakes. Adequate details are now available for moderate seismic loading, Zone 2, and are given for intermediate moment resisting frames in ACI 318-89, Chapter 21.

C108.2.8 ***UBC §1631.2.8 Anchorage of Concrete or Masonry Walls.*** Many observed failures of concrete or masonry walls in the 1971 San Fernando earthquake were attributable to inadequate anchorage between the walls and roof system. The previous observations were confirmed by research [Bouwkamp et al., 1991; Celebi et al., 1989] and the 1994 Northridge earthquake. These Requirements are added to correct this deficiency in former design practice and ensure that the wall can accommodate the loads applied between anchorages when their spacing exceeds four feet (4').

C108.2.8.1 ***Out-of-Plane Wall Anchorage to Flexible Diaphragms.*** The 1994 UBC requires that the anchorage force be increased 50 percent for only the center half of the diaphragm. In these Requirements, the C_p value of 1.2 is required for use over the full length of the flexible diaphragm. Further examination of data from instrumented buildings indicated that the increased value should be used for the full length. This is because the flexible diaphragm is a shear yielding beam, not a flexural beam. The diaphragm behavior is similar to an elastic beam spanning between shear yielding hinges adjacent to support. Third-mode response of the diaphragm also increases the anchorage force adjacent to the support of the diaphragm.

Concrete and/or masonry walls are typically anchored to the diaphragm using metal strap ties of various types embedded in the concrete and attached to wood roof framing using nails or bolts. These devices and the wood fasteners are designed using a factor of safety ranging from 1.5 to 1.7 on the allowable design value. This means that wall anchorage strengths of about (1.7) (0.3g)=0.51g do not compare well with recorded accelerations of 0.8 to 1.2g. Many examples of failed code-compliant wall anchors were noted in the 1994 Northridge earthquake.

In the 1994 UBC, C_p for these elements was increased 50 percent in the center half of the diaphragm span to 1.125, yielding a design force of 0.45 W_p in Seismic Zone 4 ($I_p = 1.0$). This provision increases C_p to 1.2, yielding a design force of 0.48 W_p in Seismic Zone 4. Using the industry standard factor of safety of 1.7, this results in a strength of 0.8 W_p for the anchorage.

Item 2. Wall anchorages are often loaded eccentrically, either because the anchorage mechanism allows eccentricity, or because of anchor bolt misalignment. This eccentricity reduces the anchorage connection capacity and hence must be explicitly considered in the design of the anchorage.

Item 3. Requires the anchorage force at pilasters to be calculated considering the wall panels between the pilasters supported on four sides. An important anchorage problem discovered by earthquake damage surveys has been the failure of the concrete at top of pilasters supporting the roof girders. When pilasters are present in the wall, the concentration of anchorages forces at the top of the pilaster must be calculated. It is customary to anchor the walls to the diaphragms assuming simple support at the ground and roof. However, when pilasters are present in the walls, their stiffening effect must be taken into account. Each panel between pilasters is supported on four sides. The reaction at the pilaster top is the result of the two-way action of the panel and is applied directly to the beam or girder anchorage to the top of the pilaster. This reaction has been found to be larger than what is provided at the girder connection, assuming the distribution of load was uniform.

Item 4. Requires a load factor of 1.7 for E in strength design for the design of embedment in concrete. The load factor for design of embedded elements in concrete is recommended as 1.7 in lieu of 1.4 as specified in UBC §1921.2.7. Use of this load factor, the load factor of 1.3 that is specified in UBC §1925.2, and the strength reduction factor of 0.65 provides an overstrength of 3.4.

Modification of the allowable stress values for connections to wood is not needed. The 1994 edition of the NEHRP *Recommended provisions* [NEHRP, 1994] recommends that the allowable stress values for connections to wood in the UBC be increased by 2.16 to represent yield strength. This factor multiplied by the difference between impact loading, which represents the duration of peak anchorage force, and the allowable increase for earthquake loading provides a combined factor of 3.25. Also, experimental data given in the *Wood Handbook* shows that the behavior of wood connectors is similar to that of a ductile material.

The allowable values given in UBC Table 21-E-1 for embedded bolts in masonry are approximately one-fifth of the expected failure load. An increase of 1.33 is allowed for seismic loads. The present design method provides a factor of 5, divided by 1.33, or 3.75 between design loading and expected ultimate load.

Item 5. Requires working stress design forces for steel elements of the wall anchorage system to be increased by a factor of 1.7. Analysis of acceleration data recorded on flexible diaphragms shows that accelerations greater than 1.0g are expected. The task force investigating tilt-up buildings damaged by the Northridge earthquake estimated a probable upper bound of 1.5g as the maximum acceleration at the diaphragm level. These investigations found many failures in the body of steel elements used in the anchorage system. A load factor of 1.7 is recommended, as the current overstrength factor of steel plate is only 1.9. This factor is calculated by using 55 ksi for ultimate stress at the net cross-section of a strap of plate, and an allowable stress of 22 ksi x 1.33. Yielding of steel elements is not expected as ultimate stress at the reduction in the area of the strap generally limits the strength. A load factor of 1.7 increases the probable overstrength to 3.2.

C108.2.9 ***Diaphragms.*** Most lateral force resisting systems are composed of two distinct parts: the vertical system that transmits lateral forces between the levels of the masses and the base of the structure, and a horizontal system that distributes lateral forces to the vertical elements.

The horizontal system may consist of a diaphragm or a horizontal bracing system. For the majority of buildings, diaphragms offer the most economical and positive method of resisting and distributing lateral forces in the horizontal plane. The economy in utilizing a diaphragm is obvious, since it is ordinarily included in a building to support vertical loads. Further detailed discussion of diaphragms can be found in Celebi et al. [1989], and in the ATC report on diaphragms [ATC-7, 1981].

A diaphragm may be regarded, in simplest terms, as a horizontal girder with several unique features. Like a girder, it is composed of connected web and flange elements. The web, or shear resisting element, is provided by the floor or roof deck; chord or boundary members serve as flanges to resist the axial tension or compression resulting from flexural action. Also, like a girder, it transfers horizontal (in plane of web) loads to its supports (vertical resisting elements). However, the girder analogy should not be regarded as complete and should only be considered as an approximation. Diaphragms have special properties compared to normal girders that must be considered in design, including:

1. The span is usually very short relative to depth; therefore, plane sections are not likely to remain plane, contrary to the usual assumption in the analysis of bending.
2. Web shear stresses and deflections due to shear are relatively more significant than stresses and deflections due to flexural action.
3. The diaphragm's components (flange, web, and connection devices) often are made of different materials. The "flanges" may be the walls normal to the direction of loading of the diaphragm, and the "flange" forces at the midspan of the diaphragm would be progressively diminished by the reduction in bending moment toward the diaphragm ends. It is intended that the chord or boundary members that resist these "flange" forces be located near the vicinity of the plane of the diaphragm.

4. Relative and absolute deflections under prescribed lateral loading are often important design limitations.

Diaphragms may be designed and constructed of many different materials: concrete, gypsum, metal, wood, combinations of materials, and other tested and approved proprietary systems. They may have flat, inclined, curved, warped, or folded configurations and may have openings.

The diaphragm may function as a very stiff element, a very flexible element or somewhere between, depending upon its physical properties and those of the vertical resisting system. Therefore, it is necessary to determine the diaphragm deflection relative to the deflections of the associated vertical resisting elements. Likewise, a knowledge of absolute deflections of diaphragms is necessary to achieve drift control and thereby reduce damage potential.

C108.2.9.1 ***Deflections.*** Concrete diaphragm deflections (both shear and flexural) may be determined by the usual mechanics of materials analyses that include all the elastic properties of the diaphragm and chord. Deflections for diaphragms of metal, wood, composite materials and proprietary items are generally established by empirical formulas determined by tests and analytical studies. The empirical formulas account for connection distortion, connection pattern, vertical load span, and other properties that contribute to the overall diaphragm flexibility and that cannot be confidently predicted by a normal analytical approach.

Particular care is needed in the design of the details of diaphragms. Connections between diaphragms and the vertical resisting elements, construction joints, the effect of openings and re-entrant angles, and continuity of boundary members are all considerations in addition to the strength and rigidity of the diaphragm. A prudent inspection program is recommended to ensure that diaphragm details will be properly constructed.

Selection of the type of diaphragm must consider, among other issues, its interaction with the vertical resisting system. In lowrise concrete or masonry buildings, deflections that can cause secondary failures in structural and nonstructural walls should be considered. For taller structures that have vertical resisting elements that will deflect significantly, it is important that a rigid diaphragm or bracing system be used to distribute the horizontal forces, since a more flexible system could permit elements of the resisting system to vibrate out of phase with each other.

As discussed in Commentary Section 105.8.2, deformations substantially larger than those corresponding to the specified design forces can occur due to major earthquake ground motion. For elements necessary for support of gravity loads, the engineer should consider the ability of these attached elements to sustain loads at deformations equal to at least $3(R_w/8)$ times the design load deformations.

C108.2.9.2 ***Diaphragm Forces.*** The actual seismic loading in the plane of a diaphragm includes the distributed inertia force equal to the response acceleration at the level of the diaphragm times its distributed mass. Equation 108-1 gives the recommended approximate method for determining the resultant of this distributed loading for the purposes of design. The prescribed lower limit prevents the effective diaphragm design force coefficient from being less than the building base shear coefficient. The details of diaphragms are generally independent of the type of lateral force resisting system selected for a building; therefore, the system R_w factor does not appear in the upper and lower limit formulas of Section 108.2.10.1.1. The upper and lower limits are based on consideration of higher mode responses at various levels of the structure and were selected consistent with the factors (C_p) assigned to other elements. The diaphragm must also be designed to transfer the concentrated shear forces from vertical resisting elements above the diaphragm to vertical resisting elements below the diaphragm whenever there are changes either in rigidities or in the lateral location of these elements. Where a discontinuity in a vertical resisting element occurs at a diaphragm level, the redistribution of the force from the element above the diaphragm to the elements below the diaphragm shall be provided for in addition to the force F_{px}.

C108.2.9.3 No Commentary provided.

C108.2.9.4 ***Diaphragm Ties.*** After the 1971 San Fernando earthquake, extensive damage was observed to low buildings having flexible diaphragms and rigid walls of concrete or masonry. The Requirements for continuous ties between the diaphragm chords and for positive anchorage between the diaphragm and walls were a result of these observations. The concept of a subdiaphragm or minidiaphragm, within the full diaphragm, has been allowed to reduce the length and number of ties required to achieve continuity between chords. As written in these Requirements, ties need only extend from the wall anchorage point to the chord of the subdiaphragm. However, there is strong evidence from recent earthquakes that this may not provide adequate development length to resist the real anchorage forces. The depth of the subdiaphragm should therefore be large enough to allow full transfer of the prescribed anchorage force by development into the sheathing. The subdiaphragm, which must meet all prescribed criteria for diaphragms, is considered to span between continuous ties of the main diaphragm or to sidewalls, but the interdependent deflection between the subdiaphragm and the main diaphragm may be ignored. The subdiaphragm must have the shear capacity to resist all tributary loads, along with loads resulting from wall anchorage. Further discussion of the subdiaphragm concept is found in the Commentary to Chapter 8, Wood.

C108.2.9.5 No Commentary provided.

C108.2.9.6 ***Plan Irregularity.*** This provision does not apply to plywood edge nailing but does apply to anchor bolts and other connections of the diaphragm. Plywood edge nailing values from UBC tables 23-I-J-1 and 23-I-J-2 may be used directly. For strength design of reinforced concrete, this Requirement may be satisfied by the use of a load factor of 1.7 for seismic load E in the load combinations given in UBC §1921.2.7.

C108.2.9.7 ***Re-Entrant Corners.*** Movement or cantilever deformation of the projecting wings of the diaphragm may be controlled by shear walls or braced frames at the ends of the wings. These resisting elements should be relatively rigid with respect to the diaphragm, such that the diaphragm wing is supported by an end wall or brace element and the parallel collector or resisting element at the re-entrant corner.

> *Exception*: The exception may be used if the three-dimensional model includes a representation of the diaphragm flexibility. A model that employs a rigid diaphragm condition will not provide the response of the projecting wings and their response amplification effects.

C108.2.9.8–C108.2.9.9 No Commentary provided.

C108.2.10 ***Framing Below the Base.*** The objective of this section is to ensure that the structural system below the base has adequate ductility and capacity to develop the forces and deformations induced by the structure above. Without such requirements, there is a possibility of major damage. For buildings utilizing special moment resisting frames, braced frames or shear wall systems, the framing from the base to the foundation must have the same degree of special detailing or have significantly greater capacity than the framing above the base. Consideration should be given to development of the full capacity of the framing above the base. Columns supporting special moment resisting frames should have the same details as the columns above the base level; and the beams of these frames at the base level should have the same details as the beams above the base. Shear walls continuing below the base should have the same boundary element details as the wall element immediately above the base. Columns below the base supporting discontinuous shear walls or braced frames are subject to the load and detail Requirements of Section 105.7.2.

Overturning effects must be carried into the foundations and the structural elements of the foundation should have the strength to develop either the capacity of the supported elements or the maximum forces that would occur in the fully yielded structural system. (See Commentary to Chapter 3 for further discussion of the foundation and related soil conditions.)

C108.2.11 ***Building Separations.*** Building separation is the distance between two adjoining buildings, or parts of the same building, with or without frangible closures. Its purpose is to permit adjoining buildings, or parts, to respond to earthquake ground motion independently. The effects of building impacts (or pounding) have been observed in all recent major and moderate earthquakes. A separation of the combined design displacements of the two units or buildings modified by $3(R_w/8)$ would appear to be appropriate until further studies are completed. Seismic separation is critical for the design of new buildings adjacent to existing ones. Since the specific dynamic behavior of the existing building usually is unknown, the engineer should be conservative in estimating its motion. Pounding is particularly important in cases where adjacent structures have diaphragms at different levels. Special care must be taken to protect elements of the lateral force resisting system from damage due to pounding and to ensure that elements potentially subject to pounding are detailed to retain vertical load-carrying capability if they are damaged.

C108.2.12 ***Boundary Members.*** The intent is that the boundary elements of the shear wall should not fail due to the deformations caused by major earthquake ground motion. Special detailing should be provided to prevent tensile fracture, and compressive crushing or buckling. Reinforced concrete shear walls have specific Requirements for the design of the boundary member, as represented by the boundary Zone (see Chapter 4).

C109 UBC §1632 Nonbuilding Structures

C109.1 General

Nonbuilding structures are structures that generally do not have the features of buildings, but come under the purview of building officials. Examples of these types of structures include tanks, pipeways, and storage racks. The specific types, however, can vary among jurisdictions. Some structures, although not buildings, have features similar to buildings. These are referred to as "building-like structures." Examples include pipe racks and multi-level equipment service platforms. Certain structures are specifically exempt from local review and therefore are not considered in this section. These include, but are not limited to: offshore platforms, wharves, electrical transmission towers, dams, and highway and railroad bridges.

C109.1.1 ***Scope.*** Nonbuilding structures are defined as self-supporting structures (i.e., not supported by other structures) that carry gravity loads and resist earthquake forces.

Many nonbuilding structures support nonstructural items that can weigh significantly more than the weight of the structure. These nonstructural items are considered flexible when their fixed base period T is greater than 0.06 seconds. Special consideration is required for these conditions.

Nonbuilding structures may be designed for resistance to earthquakes using Equation 105-1 with the R_w factors in Table 104-8. The R_w factors in Table 104-8 are based on judgment. Limiting C/R_w to a minimum of 0.40 ensures that the lateral force coefficient be at least 0.16 in Zone 4; a reasonable lower limit for the design of most nonbuilding structures. The minimum C/R_w was 0.5 in previous versions of these Requirements, and some members at SEAOC proposed reducing the value to 0.25 to allow a more rational design of flexible nonbuilding structures such as stacks and towers. Other members were uncomfortable with such a large reduction for all nonbuilding structures, especially since no constraints on calculated periods exist as they do for building structures. The 0.40 value thus represents a compromise.

Alternatively, a response spectrum analysis that includes consideration of the actual ground motion anticipated at the site may be used. The definition of the "actual ground motion at the site" may be taken as Z times the appropriate site spectrum shape in Figure 104-2 or a site specific spectrum having a 10 percent probability of exceedance in 50 years. Note that the Figure 104-2 spectrum shapes are for a 5 percent damping factor.

In general, the R_w values assigned to nonbuilding structures are less than those assigned to buildings. This is because buildings tend to have structural redundancy due to multiple bays and frame lines and contain nonstructural and nonconsidered resisting elements that effectively provide greater damping and strength during strong ground motion response. Hence, nonbuilding structures generally have lower lateral force resisting capacities than buildings. The assumed lower capacity level inherent in the values of Table 104-8, however, should not be taken for granted. The structural engineer, therefore, must evaluate the seismic response characteristics of the specific structure to determine if a more conservative R_w value is appropriate for design.

The vertical distribution of the lateral seismic forces may be determined by the static or dynamic lateral force procedures. Some nonbuilding structures are distinctly different than building structures. Consequently, determination of vertical force distribution may become very difficult. In such cases, computer methods may be the only alternative available to obtain a realistic force distribution. Such analyses will also identify critical areas where ductility demand may be concentrated, and it is essential that adequate ductility be provided at these locations. These reasons led to the exception in Section 109.5, which require irregular nonbuilding structures in Occupancy Categories I and II to be analyzed using dynamic analysis procedures, regardless of their height.

C109.1.2 ***Criteria.*** The design lateral forces for nonbuilding structures are specified at a service level, consistent with the Requirements for building structures. Future revisions to these Requirements will prescribe the minimum design lateral forces for building structures in terms of the strength or yield level, while for nonbuilding structures, it is possible that service level design forces will continue to be specified. To provide for this possible future occurrence, it was decided to specify the design lateral forces and allowables for building and nonbuilding structures separately.

C109.1.2.1 Criteria other than those included in the Requirements in Section 109 may be applicable to nonbuilding structures. The engineer is expected to review the parameters related to these criteria to ensure that they are applicable to features of the nonbuilding structures.

C109.1.2.2 Specific standards may be available for some types of nonbuilding structures that are more appropriate for the given application than those in Section 109. An approved standard is one that has been formally adopted for general use by an appropriate national organization and is based on fundamental principles of earthquake engineering practice. It should be noted that such a standard must also satisfy the limitations of Section 109.5.3. Examples of such standards include AWWA, 1984 and 1986; ACI, 1983 and 1989.

C109.1.3 ***Weight W.*** The mass of the operating contents can be substantial for many types of nonbuilding structures, and can be the major contributor to seismic loads. It is necessary, therefore, that the operating contents be considered in addition to the dead load. The mass, type, and location of contents may vary in time. The locations and masses used for design should be that distribution, from among those possible, that is most critical for the specific structure. This is to ensure that seismic performance is satisfactory for all operational use conditions of the structure.

C109.1.4 ***Period.*** The period calculated by use of Equation 105-3 is intended for buildings, and does not apply to structural systems that differ from usual building configurations and characteristics. Therefore, for nonbuilding structures, Method B procedures should be used (see Section 105.2.2), and the resulting value of C is not subject to the 84 and 80 percent limit value of C from Method A. However, the calculated period should be representative of the structural behavior and should not result in inappropriately low design loads. The fundamental period of the structural system must include consideration of the contents. It should be determined by rational methods that consider the stiffness and mass of the structure and the effective mass(es) of any operating contents. Two distinct fundamental periods may need to be determined for some nonbuilding structures. For example, with tanks, ground motion may induce sloshing of the fluid, as well as a coupled structure-fluid interaction.

C109.1.5 ***Drift.*** Drift limitations for building structures serve to control both p-delta effects and nonstructural (including glazing) damage potential. Since drift sensitive nonstructural elements are not present in most nonbuilding structures, SEAOC considers such limits as overly restrictive. Therefore, building drift limitations of Section 105.8.1 may be exceeded provided that P-delta effects of actual drifts (see Commentary Section C105.8.2) are considered in the design. However, if actual drifts are within the limits of Requirements Section 105.8.1, the P-delta effects need not be considered. For elements whose failure would cause life safety hazards, drift limits should be set to preclude element failure when subjected to the design forces and displacements multiplied by $3(R_w/8)$.

C109.2 Lateral Force

Nonbuilding structures that are similar to building structures should be designed using the appropriate R_w values of Table 104-6. The rationale for the exception regarding the use of intermediate moment resisting frames may be explained as follows: the size of nonbuilding frame structures often is not governed by traditional loadings. Rather, their size may be controlled by the footprint of the equipment they support, vibration limitations, or other operational considerations. In many cases, the resulting building-like nonbuilding structures are much stronger than required. Therefore, their ductility demand is generally much lower than a corresponding building structure. For this reason, these Requirements allow the use of intermediate moment resisting frames (IMRF) in certain applications in Zones 3 and 4, provided that R_w=4 (corresponding to low ductility demand during severe earthquake ground motion) is used for design.

C109.3 Rigid Structures

Rigid structures do not amplify the earthquake ground motions, and the response accelerations are nearly the same as the ground accelerations. The coefficient of 0.5 specified for the rigid structures in Equation 109-1 thus corresponds to an R_w factor of 2, (if the spectral acceleration is taken as the peak ground acceleration), implying that rigid structures have some inherent ductility and a capacity to absorb energy. This is reasonable considering the expected strength of rigid structures under seismic conditions. The widely used and codified standard details in the steel and concrete structures have shown inelastic deformation capability in excess of the demand.

C109.4 No Commentary provided.

C109.5 Other Nonbuilding Structures

There are a number of nonbuilding structures in common use that have special characteristics. The following guidelines address some of the items to be taken under consideration when designing these structures.

C109.5.1–C109.5.3 No Commentary provided.

C109.5.4 ***Tanks.*** Tanks are structures used for the storage of liquids. Tanks can be supported at or below grade, on skirts, on pedestals, or on braced or unbraced legs.

In the design of tanks, the lateral force V may be distributed according to the distribution of the mass. Alternatively, the effect of sloshing on mass distribution can be considered by assuming that part of the mass of the contents moves in unison with the motion of the tank, and part is assumed to move with its own frequency of vibration. This sloshing method tends to reduce the calculated lateral loads and overturning movements [U.S. AEC, 1963].

Tanks with supported bottoms founded at or below grade can be designed to resist the seismic forces as calculated in accordance with Section 109.3 or by any other approved method. All tanks with supported bottoms should be anchored to the foundation, and tanks with a height-to-diameter ratio greater than 1.5 will normally need anchor bolts to resist overturning. Often large diameter tanks need to be anchored to prevent local lifting and dropping of the wall, which can cause the deformation known as the "elephant foot" buckling. A method for determining the forces to be resisted can be found in API [1988]. The anchors need not be designed for shear forces that result from the seismic motion. The shear resistance to seismic lateral load on large tanks not subject to "walking" may be considered to be provided by the frictional resistance of the base course against the foundation, provided that the integrity of the tank and its attached piping will not be adversely damaged as a result of sliding. Piping and other appurtenances should be attached to the tank with sufficient flexibility to prevent unacceptable damage to the tank or attached appurtenance as a result of seismic induced movement such as ovaling, sliding, and/or uplift.

Tanks on pedestals and on braced or unbraced legs and their supports may be designed using Equation 105-1 in Section 105. R_w for tanks on braced or unbraced legs should be taken as 3. Horizontal tanks supported on piers can be designed using a number of approaches. For the transverse direction, Equation 109-1 may be appropriate if the piers and tank are rigid; otherwise, the use of $R_w=6$ for shear walls may be appropriate. For loads in the longitudinal direction, use of $R_w=3$ for an inverted pendulum-type structure is appropriate.

A horizontal tank should be anchored to at least one of the piers. If anchorage is provided at only one end, it would be appropriate to take "credit" for friction at the pier at the free end in determining the overturning and shear forces on the pier where anchorage is provided. Similarly, the free end should also be evaluated for shear and overturning effects due to "friction."

It is only necessary to use an I factor greater than 1.0 when the contents of the tank are acutely hazardous, safety-related (i.e. nonredundant fire water supply), or when directed otherwise by the owner of the tank. In the State of California materials that are considered acutely hazardous have been identified [OES, 1989]. For example, ammonia, benzene, and hydrofluoric acid are acutely hazardous; gasoline, and similar products are not.

C109.5.5 ***Bins and Silos.*** Bins and silos are structures used for storing solids, e.g., powder and granular materials. Silos are by definition tall bins. In bins and silos, the lateral forces are mitigated by friction in the material stored in the bin. Research has shown the equivalent viscous damping may be as much as 20 percent. Also, the center of gravity can be considered at the center of mass rather than at the centroid of the more familiar triangular distribution [ACI, 1983].

Table 104-8 assigns a value of $R_w=4$ to almost all bins, including silos, hoppers, and bunkers, but line two permits a larger value of R_w ($R_w=5$) for cast-in-place concrete silos with walls that extend down to the foundation. Even though $R_w=5$ is not intended to apply to silos supported on columns, it is applicable to silos, in which the vertical loads are carried in part by columns directly under the silo, and the seismic shear is resisted by cast-in-place concrete walls meeting the Requirements of entry 2 in Table 104-8. This higher value of R_w applies only to cast-in-place concrete silos, and does not apply to silos of other materials or to precast (staved) concrete silos. The bottom of the silo need not be at grade. The only requirement for use of $R_w=5$ is that the base shear resisting wall be continuous and be without discontinuities in strength or stiffness and not have concentrations of strength or ductility demand. $R_w=5$ should not be used if:

1. The lower silo support wall has excessive openings (i.e., greater than 20 percent of the perimeter at a given level) so as to behave like an irregular structure in a seismic event.
2. The silo does not have adequate vertical reinforcement that is properly anchored to the foundation.

C109.5.6 ***Pipe Racks.*** Pipe racks, alternately called pipe ways, are major structures that consist of a series of bents, usually interconnected, carrying one or more levels of pipe and/or electrical conduit commonly found in industrial facilities. The space beneath the pipe rack is normally left clear for vehicle and pedestrian access. They are made either of steel or concrete, the former being the most common.

In the transverse direction, if the bents are steel, they are normally designed as ordinary moment resisting frames ($R_w=6$) for the seismic lateral loads. If made of concrete, they would normally be designed as intermediate moment resisting frames ($R_w=4$).

In the longitudinal direction, pipe racks are normally braced intermittently. In these cases, an $R_w=8$ would be used. Concrete braced frames are not permitted in seismic Zones 3 and 4 (Table 104-7). Therefore, in these zones, concrete pipe racks have to be designed as intermediate moment resisting frames in both directions.

Small pipe racks may be designed as individual vertical "T" supports. In these cases, a value for $R_w=3$ is appropriate.

For earthquake loading, the bents are designed for the full contributing weight of the pipes and their contents in the transverse direction. In the longitudinal direction, a similar approach is acceptable, but may be too conservative. For a more realistic design, the friction forces between the pipes and sliding supports may be deducted in computing the longitudinal anchor support load. If the pipe is not anchored, the seismic force need not exceed the maximum pipe friction force. No more than 10 spans need be considered as tributary to a longitudinal support design. Alternatively, a dynamic analysis may be performed taking into account the fluid properties of the pipe contents.

C109.5.7 ***Structures With Heavy Loads.*** There is a wide range of structures in which the weight of the equipment is significant compared to the weight of the structure. Four discrete cases must be considered.

1. The weight of the equipment is less than 25 percent of the weight of the supporting structure, and the equipment is rigid.
2. The weight of the equipment is less than 25 percent of the weight of the supporting structure, and the equipment is flexible.
3. The weight of the equipment is greater than 25 percent of the weight of the supporting structure, and the equipment is rigid.
4. The weight of the equipment is greater than 25 percent of the weight of the supporting structure, and the equipment is flexible.

When a flexible nonstructural item with significant weight interacts with the supporting structure, it may induce amplified seismic forces in the supporting structure. Therefore, to account for these interaction effects, it is recommended that flexible equipment with weights more than 25 percent of that of the supporting structure be properly analyzed considering the dynamic interaction.

Methods for calculating the lateral load V for each of these cases are shown in Figures C109-1 and C109-2. In the fourth case, R_w may alternatively be computed using the formula:

$$R_w = \frac{R_{ws}K_sD_s^2 + R_{we}K_eD_e^2}{K_sD_s^2 + K_eD_e^2} \qquad \textbf{Eqn. C109-1}$$

where R_{ws} and R_{we} are the R_w values for the support structure and the equipment respectively, and K_s and K_e are the respective stiffnesses of the support structure and the equipment. D_s and D_e are the relative displacements at the center of mass of the support structure and the equipment respectively, resulting from a force distribution given by the equation:

$$F_x = \frac{W_xh_x}{\sum_i W_ih_i} \qquad \textbf{Eqn. C109-2}$$

C109.5.8 ***Storage Racks.*** Storage racks are typically mass-produced elements that are assembled on site. The geometry and strength requirements are quite varied, and standard components are mixed appropriately to provide the required performance. Many racks are made of relatively slender or cold formed elements, which require special evaluation procedures. Machine-loaded storage racks are typically quite heavy and are supported by a slab-on-grade. Racks supported by a building structure are by definition not nonbuilding structures, and should either be designed using the Requirements of Section 107, or should be carefully investigated for dynamic interaction, preferably in conjunction with the design of the building itself. This Commentary does not address storage "rack-supported buildings." "Rack-supported buildings," are essentially racks with cladding and a roof added. The normal procedure is to design these structures as storage racks with due consideration of the unique seismic behaviors possible as noted below, as well as normal environmental loads such as wind and snow.

The design of free standing (i.e., not supported by another structure that filters the seismic accelerations) storage racks per Section 109 is appropriate for the typical configuration of selective, drive-in, and cantilever racks if W is determined by including at least two-thirds of the rated capacity in the base shear computation. This is an increase with respect to previous versions of these Requirements, reflecting the relatively poor performance of some racks in recent earthquakes. In addition, storage rack design provisions were originally intended for racks located in low occupancy warehouses. The increased use of racks in stores open to the public suggests more conservative provisions are appropriate. Note that some reduction in live load is permitted, unlike for most other nonbuilding structures, where W includes all normal operating contents. This reduction acknowledges that it is typically unlikely that all levels of a rack will be loaded simultaneously in all bays. If such loading is possible, 100 percent of the live load should be used. Some reduction in load is also believed appropriate because the stored items are not typically attached to the supporting support beams, and it is judged that the total live load will not contribute to any response at the fundamental frequency of the rack system.

The discussion of the low R_w value for nonbuilding structures relative to building structures due to low redundancy does not apply to all installations, some of which are highly redundant. Often racks are arranged in back-to-back rows with a large number of bays in the longitudinal direction. Certainly these racks are highly redundant in the longitudinal direction, and have some redundancy in the transverse direction, depending on how the adjacent rows are interconnected. However, the same rack components can be used to assemble a single bay, single row rack that has no redundancy.

The low R_w value reflects the lowered ductility of the typical rack structure, which is often snapped together and braced with slender cold-formed elements with little inelastic reserve. Connection strength and stiffness are usually determined by testing, and are fairly consistent within types. The behavior of these unique elements must be considered in any dynamic analysis. For this reason, a storage rack is almost always a Type 2 or 3 structure (or semi-rigid, as defined by UBC §2250). Unusual or slender racks should be investigated more carefully, particularly for P-delta effects, which can be considerable in Zones 3 and 4 when unsupported lengths exceed about 40 times the column dimension perpendicular to the bending axis.

As an acceptable alternative to the use of the Requirements of Section 109.5, storage racks can be designed using UBC §2231 et seq., Div. VI, subject to the restriction that the value of C/R_w not be less that 0.5. The standard allows for the design of racks that are interconnected such that there are a minimum of four columns in any direction on each column line using a reduced load of 50 percent of the rack-rated capacity. In order to take advantage of this reduction, the connection between columns of adjacent racks must be capable of resisting the shear required to provide coupled flexural action of the racks.

The engineer should be cognizant of the fact that storage racks are easily reconfigured, which may result in configurations substantially different than those intended by the engineer. The engineer should also recognize that a typical owner may not be sophisticated enough to accurately estimate the loads to which the racks may be subjected. Some conservatism in design loads may be appropriate.

Past storage rack failures in earthquakes have resulted from poor construction procedures and overloading. Poor installation of expansion anchors is common. In large installations, especially in areas where public access is permitted, the engineer should consider establishing a testing and/or inspection program for bolts required to act in tension.

C109.5.9 ***Vertical Vessels.*** Ductile response for vertical vessels is achieved through inelastic stretching of the anchor bolts. In order to ensure that ductile response is achieved by way of anchor bolt stretching, the anchor bolts should be designed so that the concrete embedment is substantially stronger than the forces required to stretch the bolt. A procedure such as described in Appendix B of ACI 349

[ACI, 1985] may be used to design anchor bolt embedments to ensure full bolt capacity and a ductile failure mode. The bolt lengths and patterns should be designed so that the total energy of bolt stretching beyond yield under seismic and other forces is consistent with the energy absorption level needed to achieve the specified level of ductility for the overall structure. The integrity of the bolts should be checked for deformations at least $3(R_w/8)$ times those developed under design forces.

C109.5.10 ***Other Structures.*** Some nonbuilding structures, such as concrete pedestal type structures, may inherently be so strong in some cases that they will remain in the elastic range, even during maximum ground motions anticipated at the site. The engineer may use an ordinary space frame design, provided an $R_w = 1$ is used for the design loading by the same logic that permits IMRF. However, it is recommended that the engineer always include some ductile capacity. In this case, it is necessary to provide for continuity and development of longitudinal reinforcing and to provide sufficient transverse reinforcement to avoid brittle shear and/or development failures.

C110 UBC §3400 Existing Structures

There are no clear criteria in the UBC on how to repair buildings and structures damaged in earthquakes. Building officials are left with either having to interpret "repairs" in the UBCs definition section to include damage from the event, or taking the time to develop a repair ordinance for adoption by their jurisdiction. According to UBC §219, repair is defined as, ". . . the reconstruction or renewal of any part of an existing building for the purpose of its maintenance." This provision is difficult to apply to a building damaged by a disaster. Further, UBC §3403 does not give guidance on the repairs required.

When federal assistance is available, federal regulations require that repair requirements be adopted and in place prior to the beginning of work so that the assistance will apply to the developed criteria. If the requirements are not put in place in a timely fashion, federal assistance provided will be limited to simply restoring the building to its pre-event condition. Repairs to this level simply leave the building vulnerable to repeat damage from future events.

The criteria contained in these Requirements are based on a memorandum of understanding between the California Office of Emergency Services (OES) and the Federal Emergency Management Agency (FEMA) developed shortly after the Northridge earthquake. By this memorandum, FEMA agreed to fund repairs to public buildings to this level, as a minimum, unless the jurisdiction had adopted by ordinance other standards for repair. In many cases, jurisdictions accepted this criteria and eliminated the need to pass specific repair ordinances.

C111 Future Objectives and Research Requirements

As a result of the deliberations involved in the formulation of these Requirements, the following items were determined to be essential for future editions of this document.

1. A strength design basis is already in effect for reinforced concrete, but is also necessary for steel, masonry, and, possibly, wood.
2. Description and specification of earthquake ground motion in the form of contour maps for site intensity measures, site-dependent spectra, and representation of duration of ground motion effects. The contours are to be based upon a consistent risk basis. Zoning considerations based on regional design and construction practices should be well described and justified.
3. A dynamic analysis procedure without the necessity of specified base shear scaling of results. A two-level spectrum approach for damage limitation and collapse prevention may be considered.
4. Procedures for R_w factor evaluation that include recognition of system redundancy and uncertainty of performance.
5. Coordination with mechanical and electrical engineering professions for seismic design of equipment.
6. Further research on: behavior of large steel sections and connections; panel zone yield effects; and strong girder-weak column effects.
7. For reinforced concrete: resolution of the shear stress provisions for beam-column joints; a review of hooked bar anchorage requirements with respect to strain hardening under cyclic load reversals; an overall review of confinement requirements with objectives to reduce clutter and ensure quality of the placed concrete.
8. Period—definition and procedures for evaluation. Braced frames require a period equation that represents bracing configuration.
9. Improved design provisions for dual systems.
10. Flexible diaphragm effects on period.
11. Standards for dynamic tests—cyclic loadings at appropriate levels and numbers of cycles representative of structural response due to the maximum expected intensity and duration of ground motion.

C112–C149 Reserved

C150 UBC §1650 General Requirements for Seismic-Isolated Structures

C150.1 Introduction

Seismic isolation, commonly referred to as base isolation, is a design concept based on the premise that a structure can be substantially "decoupled" from potentially damaging earthquake ground motions. By decoupling the structure from ground shaking, isolation reduces the level of response in the structure from the level which would otherwise occur in a conventional, fixed-base building. It is assumed that decoupling will be accomplished using an isolation system that makes the effective period of the isolated structure several times greater than the period of the structure above the isolation system.

The potential advantages of seismic isolation and advancements in isolation system products has lead to the design and construction of a number of isolated buildings and bridges in the last decade. This has created a need to supplement existing codes with design requirements developed specifically for such structures. This need is shared by the public, which requires assurance that isolated buildings are "safe" and by the engineering profession, which requires a minimum standard upon which design and construction can be based. Accordingly, the SEAOC Seismology Committee has developed these Requirements for isolated structures to supplement the Requirements for fixed-base structures.

The seismic isolation concept and criteria that would be appropriate for design and construction of isolated structures have been considered by various SEAOC committees, active since the early 1980s. In 1986, the first collection of design requirements for base isolated structures, *Tentative Seismic Isolation Design Requirements* [SEAONC, 1986], were published. Previous Requirements were developed using the same seismic criteria as the 1990 Blue Book, and incorporated similar design concepts, such as the prescription of minimum design forces and displacements by simple formulas.

Recognizing the need for a document that would represent a consensus opinion of all sections of SEAOC, the Seismology Committee formed the Ad Hoc Base Isolation Subcommittee in early 1988 to synthesize a single seismic isolation design document from existing material. Accordingly, representatives of the Northern, Southern, and Central Sections of SEAOC developed design requirements entitled: "General Requirements for the Design and Construction of Seismic-Isolated Structures." Those Requirements were approved by the Seismology Committee and published as Appendix 1L of the 1990 Edition of the Blue Book [SEAOC, 1990]. Those Requirements were also adopted (with minor editorial changes) by the International Conference of Building Officials (ICBO) and published as a nonmandatory appendix to Chapter 23 of the 1991 Edition of UBC [ICBO, 1991]. Improvements to the 1990 edition of the Requirements for isolated structures have been incorporated by the Seismology Committee into this edition of the Blue Book, and similar revisions have been adopted by ICBO for the 1994 edition of the UBC [ICBO, 1994].

C150.2 General

The underlying philosophy guiding the development of these Requirements for isolated structures may be characterized as a combination of SEAOC's primary performance objective for fixed-base buildings, which is the protection of life safety in a major earthquake, and the additional performance objective of damage protection, an attribute provided by isolated structures. The design criteria of the isolation Requirements are based on this combination of life safety and damage protection goals. These criteria are summarized as follows:

1. These Requirements specify two levels of earthquake: the design basis earthquake (DBE) and the maximum capable earthquake (MCE). DBE is the same level of ground shaking as that recommended for design of fixed-base structures. The MCE is a higher level of earthquake ground motion defined as the maximum level of ground shaking that may be expected at the building site within the known geological framework.
2. These Requirements state that isolators must be capable of sustaining loads and displacements corresponding to the MCE without failure.
3. These Requirements state that the structure above the isolation system remain "essentially elastic" for the DBE.

The performance objectives and design requirements for fixed-base and isolated buildings vary significantly. The performance objective for fixed-base construction is *life safety* in a design earthquake; the intent is to prevent substantial loss of life rather than control damage. For isolated buildings, the performance objectives are:

1. Minimal to no damage in the design earthquake (thus providing *life safety*).
2. A stable isolation system in the maximum capable earthquake.

The performance of an isolated building in a design earthquake will likely be much better (less interstory drift, smaller floor accelerations) than its fixed-base counterpart. Further, isolated buildings can be designed to provide continued function following a design earthquake; a level of performance that is very difficult to achieve with conventional fixed-base construction.

Fixed-base buildings are generally designed using large response modification factors to reduce elastic spectral demands to a design level, a strategy predicated on significant damage to the framing system. Such buildings are checked for response in the design earthquake only; there is no design check for the maximum capable earthquake. In contrast, isolated buildings are designed using a dual level approach, namely, the framing system is designed to remain essentially elastic (no damage) in the design earthquake, and the isolators are designed (and tested) to remain stable in the maximum capable earthquake.

These Requirements are intended to be applicable to most types of isolation systems currently in use. However, these Requirements may not apply to nontypical isolation systems. In such cases, it is recommended that a complete and detailed nonlinear dynamic analysis be performed, and that such analysis and resulting designs be subjected to independent design review per §C160 (UBC §1600).

C151 Definitions for Seismic-Isolated Structures

C151.1 Isolation System/Interface

Isolation system terminology is shown in Figure C151-1. Isolation interface is assumed to be a boundary between the upper portion of the building, which is isolated, and the lower portion, which is rigidly attached to the foundation or ground. The isolation interface can be assumed to pass through the mid-height of elastomeric bearings or the sliding surface of sliding bearings. The isolation interface need not be a horizontal plane, but could change elevation if the isolators were positioned at different elevations throughout the building.

The isolation system includes the isolator units, connections of isolator units to the structural system, and all structural elements required for isolator stability. Isolator units include bearings that support the building's weight and provide lateral flexibility. Typically, isolation system bearings provide damping and wind restraint as integral part of the bearing. Isolator systems may also include supplemental damping devices.

Structural elements that are required for structural stability include all structural elements necessary to resist design forces at the connection of the structure to isolator units. For example, in Figure C151-1, the column segment and beam immediately above the isolator are shown as elements of the isolation system because they are necessary to resist forces due to the lateral earthquake displacement of the isolators.

C151.2 Effective Stiffness and Damping

Typically, isolation systems are nonlinear and the effective stiffness of the isolation system is displacement- and velocity-dependent. Idealized force-deflection relationships for seismic isolators are shown in Figure C151-2. Note that the maximum and minimum forces do not necessarily correspond to the maximum and minimum displacements, respectively, contrary to the assumptions currently made in these Requirements.

The effective stiffness k_{eff} of a seismic isolator is calculated using the forces in the isolator at the maximum and minimum displacements (Equation C151-1).

$$k_{eff} = \frac{F^{+} - F^{-}}{\Delta^{+} - \Delta^{-}} \qquad \textbf{Eqn. C151-1}$$

For isolators whose properties are independent of velocity, the forces in the isolator at the maximum and minimum displacements will generally be maximum and minimum forces, respectively. For isolators whose properties exhibit velocity dependence, the forces in the isolator at the maximum and minimum displacements will generally be less than the maximum and minimum forces, respectively. These Requirements assume that maximum and minimum forces in an isolator are realized at maximum and minimum displacements, respectively. For most types of isolator, this assumption is reasonable.

For the purpose of design, energy dissipation is characterized as equivalent viscous damping. Equation C151-2 defines the equivalent viscous damping (β) for a single isolator; this equation is correct for true viscoelastic behavior but is only an approximation for hysteretic behavior. In this equation, *Total Area* is the area enclosed by the force-displacement loop of a single isolator in a complete cycle of loading to displacement D. The effective stiffness (k_{eff}) of the isolator at displacement D is calculated using Equation C151-1.

$$\beta = \frac{1}{2\pi}\left[\frac{Total\ Area}{k_{eff}\,D^2}\right]$$ **Eqn. C151-2**

C152 UBC §1652 Symbols and Notations for Seismic-Isolated Structures

The definitions of design displacement, total design displacement, and total maximum displacement, are depicted in Figure C151-3.

C153 UBC §1653 Criteria Selection for Seismic-Isolated Structures

C153.1 Basis for Design

Seismic isolated buildings may be more susceptible than conventional buildings to significant changes in response due to: 1) vertical ground motions, and 2) stiffness and strength degradation. As such, items 1 and 2 above should be considered in the analysis and design of seismic isolated buildings.

C153.2 Stability of the Isolation System

These Requirements specify verification of the stability of the isolators for the MCE displacement. Analysis is required to determine the maximum and minimum vertical loads on isolators, and tests of prototype isolators are required to verify the stability of isolators for the calculated loads at the MCE displacement.

C153.3 Occupancy Categories

The importance factor, *I*, for a seismic-isolated building shall be taken as 1.0, regardless of the occupancy category.

C153.4 Configuration Requirements

Each structure shall be designated as either regular or irregular on the basis of the type of structural framing above the isolation system.

C153.5 Selection of Lateral Response Procedure

Three procedures are addressed in these Requirements: static analysis, response spectrum analysis, and time-history analysis. The static analysis procedure is generally used to start the design process and to calculate benchmark values for key design parameters (displacement and base shear) evaluated using either response spectrum or time-history analysis procedures.

The static analysis procedure is straightforward. Correct use of this procedure will generally produce reasonable estimates of design forces and displacements. The procedure is based on a spectral shape proportional to $1/T$, consistent with the constant velocity domain. The static procedure cannot be used when the spectral demands may not be adequately characterized using the assumed spectral shape, namely, for:

1. Isolated buildings located in the near-field.
2. Isolated buildings on soft soil sites.
3. Long-period isolated buildings (beyond the constant velocity domain).
4. For buildings located in regions of low to moderate seismicity (for which the assumed spectral shape may not apply).

Further, the static procedure cannot be used for highly nonlinear isolation systems. The restrictions placed on the use of the static procedure for design preclude its widespread use in California.

Response spectrum analysis is permitted for the design of all isolated buildings except for those buildings located on very soft soil sites (for which site-specific spectra should be established) and buildings supported by highly nonlinear isolation systems for which the assumptions implicit in the definitions of effective stiffness and damping break down.

Time-history analysis is the default analysis procedure; it must be used when the restrictions set forth for static and response spectrum analysis cannot be satisfied and may be used for the analysis of any isolated building. Arguably the most detailed of the analysis procedures, the results of time-history analysis must be carefully reviewed to avoid gross over- or under-design.

C154 UBC §1654 Static Lateral Response Procedure for Seismic-Isolated Structures

C154.1 No Commentary provided.

C154.2 Deformational Characteristics of the Isolation System

The deformational characteristics of the isolation system determine: 1) the design displacements and 2) the maximum forces transmitted to the isolated structure. Deformational characteristics are represented by the effective (secant) stiffness of the isolation system at the response amplitude of interest. Recognizing that force-displacement hysteresis of an isolation system may change over the course of an earthquake, the maximum effective stiffness is used to calculate the maximum force transmitted by the isolators, and the minimum effective stiffness is used to evaluate the fundamental period of the isolated building (likely producing conservative estimates of the design displacement). The limiting values are generally established during the design phase and are required to be confirmed by prototype testing.

C154.3 Minimum Lateral Displacements

C154.3.1 ***Design Displacement.*** Equation 154-1 prescribes the design displacement, D, at the center of mass of the isolated building. This equation is based on the assumptions: 1) that the framing above the isolators is rigid; and 2) the centers of mass and stiffness of the isolated building are coincident. Equation 154-1 is derived as follows.

The spectral shape is inversely proportional to the period. The 5 percent damped spectral ordinates are obtained by anchoring the spectral shape to the product of Z, N, and S_I at a period of T second. The base shear is calculated at the isolated period, T_I, assuming 100 percent of the building mass participates in the isolated mode. The 5 percent damped base shear is reduced by a damping factor, B, to reflect the effective damping, β, in the isolation system, resulting in a base shear, V_I:

$$V_I = \frac{ZNS_I}{BT_I} W_I \qquad \textbf{Eqn. C154-1}$$

where W_I is the weight of the superstructure.

The base shear given by Equation C154-1 is converted into a design displacement, D, through the simple relation between pseudoacceleration (expressed herein as base shear divided by the reactive mass) and spectral displacement (equal to the design displacement in this instance):

$$D = \frac{V_I}{(W_I/386.4)}\left(\frac{T_I}{2\pi}\right)^2 = \frac{386.4ZNS_IW_I}{W_IBT_1}\,\frac{T_I}{4\pi^2} \cong \frac{10ZNS_IT_I}{B} \qquad \textbf{Eqn. C154-2}$$

The near-field factor, N, reflects new knowledge regarding seismic demands close to active faults. The N factor is considered to be both a function of the distance from the site to the active fault and the magnitude of the maximum capable earthquake, M_{MCE}, that could occur on that fault. Both the location and M_{MCE} magnitude of faults must be based on properly substantiated data, such as that published by the United States Geological Survey or the California Department of Mines and Geology.

The damping factor, B, is used to reduce the displacement response of an isolation system. This factor is a function of the amount of damping afforded by the isolation system (measured as a percentage of critical damping, β). The values for the damping factor are based on the median spectrum amplification factors in the velocity domain as given in *Earthquake Spectra and Design* (Newmark and Hall, 1982). As shown in Table C154-1, the damping factor for values of β exceeding 20 percent include a slight (i.e., conservative) reduction with respect to the spectral amplification factors of Newmark and Hall [1982]. Caution should be exercised in using values of β exceeding 20 percent of critical if the isolators exhibit true viscous damping. In this instance, the viscous damping forces should be calculated and appropriately added to the hysteretic forces in the isolation system.

C154.3.2 ***Isolated Structure Period.*** The period, T_I, of the isolated structure is defined by Equation C154-3 in terms of the minimum effective stiffness of the isolation system k_{min} as follows:

$$T_I = 2\pi\sqrt{\frac{W_I}{k_{min}g}}$$ **Eqn. C154-3**

where g is the gravity constant (e.g., 386 in./sec.^{2}).

C154.3.3 ***Total Design Displacement.*** The design of isolated structures must consider additional displacement due to actual and accidental eccentricity, similar to the additional loads prescribed for fixed-base structures. Equation C154-4 provides a simple means to combine translational and torsional displacement in terms of the gross plan dimensions of the building (i.e., dimensions b and d), the distance from the center of the building to the point of interest (i.e., dimension y) and the actual plus accidental eccentricity, as follows [Kelly, 1993]:

$$D_T = D\left[1 + y\frac{12e}{b^2 + d^2}\right]$$ **Eqn. C154-4**

where e is equal to the sum of the actual eccentricity and the accidental eccentricity.

Equation C154-3 factors the design displacement, D, at the center of the building to account for additional displacement at the corners or edges of the building due to torsion, assuming that the stiffness of the isolation system is distributed in plan proportional to the distribution of the supported weight of the building. The reader is referred to Kelly [1993] for additional information.

Smaller values of D_T can be used for design if the isolation system is configured to resist torsion (e.g., stiffer isolator units are positioned near the edges and corners of the building). The minimum value of D_T is set equal to $1.1D$ for all types of isolation systems.

C154.3.4 ***Total Maximum Displacement.*** The static analysis procedure and related dynamic analysis procedures characterize the minimum MCE response by multiplying the DBE response by a factor M_M. The values for M_M are given in Table 152-2 (see Requirements) for Seismic Zones 3 and 4.

C154.4 Minimum Lateral Forces

C154.4.1 ***Isolation System and Structural Elements at or Below the Isolation Interface.*** The design actions for elements at or below the isolation interface (see Figure C151-1) are based on the maximum forces delivered by the isolation system during the design basis earthquake.

The maximum force, V_b, is the product of the maximum stiffness of the isolation system, k_{max}, and the design displacement, D. The design force, V_b, is divided by 1.5 in Equation 154-5 to reduce the strength level forces to the allowable stress design (ASD) or working stress design (WSD) level. The divisor of 1.5 is a compromise between the seismic load factor for reinforced concrete of 1.40 and the calculated seismic load factor for structural steel of 1.67 (assuming that the one-third increase for seismic forces is not applied).

Equation 154-5 was developed specifically for use in regions of high seismicity (Zone 4) wherein the difference between the total design displacement and total maximum displacement, M_M, is relatively small; that is, if a supporting element was designed for design basis earthquake forces at the strength level, it is probable that such a supporting element could resist the forces associated the maximum capable earthquake without failure.

There are significant differences in values of M_M between regions of high and low seismicity: values of M_M may be less than 1.25 in regions of high seismicity, but may exceed 2.5 in regions of low seismicity. As such, in a region of low seismicity, a supporting element designed for design basis earthquake-induced forces may be unable to sustain forces associated with the maximum capable earthquake without significant distress or failure.

C154.4.2 ***Structural Elements Above the Isolation System.*** The design of the framing above the isolation system is based on the maximum force delivered by the isolation system divided by a response reduction factor, R_{wI}. The values assigned to R_{wI} are approximately one-third to one-quarter of the values assigned to conventional construction. By using relatively small values for R_{wI}, a significant measure of damage control is afforded in the design earthquake.

C154.4.3 ***Limits on*** V_s**.** Three lower bound limits are set on the minimum seismic shear to be used for the design of the framing above the isolation system. The first limit requires design base shear to be at least that of a fixed-base building of comparable period. The second limit ensures that the elements above the isolation system remain elastic during the design wind storm. The third limit is designed to prevent the elements above the isolation system deforming inelastically before the isolation system is activated.

C154.5 Vertical Distribution of Force

The vertical distribution of the seismic base shear is similar to that used for fixed-base buildings, namely, a distribution that approximates the first mode shape of the fixed-base building. This distribution conservatively approximates the inertia force distributions measured from time-history analyses.

C154.6 Drift Limits

The maximum interstory drift ratio limit of $0.010/R_{wI}$ for isolated structures is consistent with the corresponding drift ratio limit of $0.040/R_w$ for fixed-base buildings since the ratio R_w/R_{wI} is approximately 4 for common framing systems.

C155 UBC §1655 Dynamic Lateral Response Procedure for Seismic-Isolated Structures

C155.1 General

The restrictions placed on the use of the static lateral response procedures (Section 153.5.1) effectively require dynamic analysis for most isolated structures. However, lower bound limits on design displacements and design forces are specified in Section 155 as a percentage of the values prescribed by the static procedure. These lower bound limits on key design parameters ensure consistency in the design of isolated structures and serve as a safety net against gross underdesign. Table C155-1 provides a summary of the lower bound limits on dynamic analysis specified by these Requirements.

C155.2 Isolation System and Structural Elements at or Below the Isolation Interface

The limits set on key design parameters are based on the design displacement, D. This displacement reflects the response of a single degree of freedom system to the prescribed seismic input. If the isolated superstructure is rigid, the roof displacement, isolator displacement, and design displacement are all equal. If the isolated superstructure is flexible, the roof displacement is greater than the design displacement, and the isolator displacement, D' is less than the design displacement. Equation 155-1 relates the isolator displacement to: 1) the design displacement, 2) the period of the fixed-base building, T, and 3) the period of the isolated building. The reader is referred to Winters and Constantinou (1993) for the details of this derivation.

C155.3 No Commentary provided.

C155.4 Ground Motion

C155.4.1 ***Design Spectra.*** Site-specific spectra must be prepared for the design of long-period base isolation systems, base isolated buildings on soft soils, and base isolated buildings located either near an active fault or in lower seismic zones. This Requirement for site-specific spectra stems from the uncertainties associated with the standard spectral shapes set forth in these Requirements and in the UBC.

The standard design spectrum is developed by scaling the normalized spectrum of Figure 104-2 (for the appropriate soil type) by the seismic zone factor, Z, and the near-field factor, N. Values for N range currently between 1.0 and 1.5, although there is evidence to support a larger value of N for sites adjacent to faults capable of generating large magnitude earthquakes. The value assigned to N is dependent on:

1) the distance from the building site to the causative fault, and 2) the magnitude of the largest possible event (maximum capable earthquake) on the causative fault.

The development of a site-specific spectrum for the design basis earthquake is encouraged by these Requirements. However, the ordinates of the spectrum are not permitted to be less than 80 percent of the ordinates of the design spectrum to guard against the use of inappropriately generated site-specific spectra. (A similar limit is imposed on the ordinates of a site-specific maximum capable earthquake spectrum.)

A design spectrum must be constructed for the maximum capable earthquake (MCE). The MCE spectrum may be taken as the spectral shape of Figure 104-2 scaled by the product of M_M, Z, and N. Values for M_M are given in Table C152-2 for Seismic Zones 3 and 4. A site-specific MCE spectrum must be developed for other seismic zones.

C155.4.2 ***Time Histories.*** Rules for the development of ground motion records for analysis of isolated buildings are provided in these Requirements. The use of recorded ground motion component pairs is encouraged to retain the phasing between the components.

Arbitrary scaling of ground motion records in amplitude or frequency so as to achieve specified spectral ordinates should be avoided. Ground motion pairs that: 1) are consistent with magnitude and source characteristics of the design (or maximum capable) earthquake, and 2) require minimal amplitude and frequency scaling, should be used for analysis and design.

C155.5 Mathematical Model for Seismic-Isolated Structures

C155.5.1 ***General.*** Several modeling procedures have been developed for the analysis and design of seismic-isolated buildings. These procedures are described briefly below. None of the procedures can adequately capture the secondary forces that develop as a function of the horizontal displacement (often large) of the isolators. One key example is the moment, equal to the product of the load on the isolator and the isolator displacement (known to many as the P-delta effect), that must be resisted by the isolator, the connections of the isolator to the structural framing above and/or below the isolator, and the structural framing above and/or below the isolator.

Three-dimensional elastic models of isolated buildings are commonly used for response spectrum analysis. The model of the superstructure should include all significant structural members in the building frame and accurately account for their stiffness and mass. The isolators are modeled as linear springs with stiffness equal to the effective stiffness, requiring an a priori estimate of the likely displacement in the isolators.

Another common procedure used for the design of isolated buildings assumes nonlinear isolators and a linear elastic superstructure. The procedure is appropriate for buildings in which the superstructure is assumed to undergo none to minimal inelastic response. This procedure requires the development of a three-dimensional elastic model of the superstructure (Model 1) and a three-dimensional model of the isolated building (Model 2). For Model 2, the spatial distribution and nonlinear characteristics of the isolators are modeled explicitly, and the superstructure is modeled as a lumped-mass stick model, with the stiffness and mass (at 3 degrees of freedom per floor level) of the stick model being determined by the static condensation of the three-dimensional elastic model of the superstructure. The execution of this procedure involves a number of steps, namely:

1. Create Models 1 and 2.
2. Analyze Model 2 using nonlinear time-history analysis to determine maximum displacements in the isolators and maximum nodal inertia forces in the superstructure.
3. Use the displacements of step 2, above, to determine the design displacements in the isolators.
4. Impose the maximum nodal inertia forces at the master degrees of freedom in Model 1 and calculate design actions for the superstructure, and the maximum and minimum earthquake loads on individual isolators.
5. Use the displacements of step 3, above, and maximum forces of step 4, above, to determine the design displacements and forces on the individual isolators.

The recent development of analysis software packages that include three-dimensional elastic modeling of the superstructure and three-dimensional nonlinear modeling of the isolators will simplify the execution of the above procedure.

Another procedure involves the development of a complete three-dimensional nonlinear model of the building. This level of effort is computationally intensive and likely rarely justified. This procedure should probably be used only for isolated buildings in which the superstructure is likely to experience substantial inelastic response–an assumption at odds with the stated performance goals for seismic-isolated buildings.

C155.6 Description of Analysis Procedures

C155.6.1–C155.6.2 No Commentary provided.

C155.6.3 ***Response Spectrum Analysis.*** The isolated building should be represented by a three-dimensional, linear elastic structural model. The isolators should be represented by linear springs with stiffness k_{eff}. (The calculation of k_{eff} may require multiple iterations, as k_{eff} will be a function of the target displacement.) The first mode damping ratio should be set equal to β for the isolation system. The reader is referred to Kelly [1993] for additional information.

C155.6.4 ***Time-History Analysis.*** Time-history analysis shall be performed with at least three matched pairs of horizontal time-history components, as defined in Section 155.4.2. Parameters of interest should be calculated for each time-history analysis pair. The parameters of interest should include member forces, connection forces, interstory drift, isolator displacements, and overturning forces.

Each matched pair of horizontal ground motion records should be simultaneously applied to the mathematical model, considering the most disadvantageous location of mass eccentricity, to calculate the maximum displacements in the isolation system. Common design practice is to 1) impose the orthogonal components along each of the principal axes of the building (two analyses), and 2) repeat step 1 after changing the sign of the ground motion components (two analyses), for a total of four analyses per matched pair, per mass eccentricity.

To reduce the computational effort, preliminary analysis may be undertaken to identify: 1) the most disadvantageous location of the mass eccentricity, 2) the critical matched pair of ground motion records, and 3) the critical orientation of the matched pair identified in Item 2, above.

Much of the analysis and design work may be completed with this substantially reduced set of parameters. Once the analysis and design effort is near completion, the final design(s) should be analyzed using the unreduced set of parameters.

C155.7 Design Lateral Force

C155.7.1 ***Isolation System and Structural Elements at or Below the Isolation Interface.*** See Commentary Section C154.4.1.

C155.7.2 ***Structural Elements Above the Isolation System.*** See Commentary Section C154.4.2.

C155.7.3 ***Scaling of Results.*** Table C155-1 (Commentary Section C155.1) provides a summary of the lower bound limits on the results of dynamic analysis that can be used for design.

C156 No Commentary provided.

C157 UBC §1657 Detailed Systems Requirements for Seismic-Isolated Structures

C157.1–C157.2.4 No Commentary provided.

C157.2.5 ***Displacement Restraint.*** The use of a displacement restraint system is not encouraged by these Requirements. Should a displacement restraint system be implemented, explicit analysis of the isolated building accounting for the displacement restraint system is mandated for earthquake demand based on the MCE.

C157.2.6 ***Vertical Load Stability.*** The design vertical load to be used in checking the stability of any given isolator shall be calculated using the maximum *DL* and *LL* and the peak MCE earthquake-induced axial load on that particular type of isolator.

C157.2.7–C157.2.8 No Commentary provided.

C157.2.9 ***Quality Control.*** The implementation of a rigorous quality control program (such as that assigned an ISO rating) is key to the production of isolators of uniform quality with consistent mechanical properties. This quality control program should be implemented for both prototype and production isolators. If the production (quality control) testing results are to be based in any part on the results of the prototype tests, the production testing program should be completed on each of the prototype isolators prior to starting the prototype tests.

C158–C159 No Commentary provided.

C160 UBC §1660 Design and Construction Review for Seismic-Isolated Structures

Design review of 1) the analysis and design of the isolation system, and 2) the isolator testing program, is mandated by these Requirements for two key reasons:

1. The consequences of isolator failure could be catastrophic. As such, detailed review is warranted.
2. Isolator design and fabrication technology is evolving rapidly, perhaps utilizing technologies unfamiliar to many design professionals.

Requirements Section 160.1 requires that such review be performed by 1) a team independent of the design team and the project contractors, and 2) that the review team be composed of individuals with special expertise in one or more aspects of the design and implementation of seismic isolation systems. The review team should be formed prior to the development of ground motion criteria and isolator design options. Further, the review team should be given complete access to all pertinent information such that the review team can work closely with all consultants and regulatory agencies involved in the project.

C161 UBC §1661 Required Tests of Isolation System for Seismic-Isolated Structures

C161.1 General

All isolator testing should be witnessed and reported by a qualified, independent inspector.

C161.2 Prototype Tests

C161.2.1 No Commentary provided.

C161.2.2 ***Record.*** For each cycle of test the force-deflection behavior of the prototype test specimen must be recorded so that the data can be used to determine whether the isolation system complies with both these Requirements and the specification prepared by the engineer of record. The engineer of record and the independent review team (see Section 160) should review all raw data from the prototype tests.

C161.2.3 ***Sequence and Cycles, Item 4.*** The total number of testing cycles specified in Item 4 is related to the site coefficient and isolation system damping, because the number of cycles of substantial response will likely be greater for soft sites and systems with small damping values.

C161.2.4 ***Units Dependent on Loading Rates.*** If the mechanical characteristics of the isolation system are dependent on the rate of loading, additional dynamic tests must be performed to characterize this dependence. Rate-dependence behavior will be exhibited by most sliding isolation systems (velocity-dependent) and selected elastomeric isolation systems (strain rate-dependent). Reduced-scale models of isolators can be used to capture rate effects on stiffness and damping values, provided that the reduced-scale isolators are fabricated using the same processes and quality control procedures as the full-size isolators. Dimensional relationships between full and reduced scale units should be established and verified prior to finalizing the testing program.

The criteria to be used to judge whether the properties of an isolation system are dependent on the rate of loading are specified, namely, the isolators are to be considered rate-dependent if the test data demonstrates that the effective stiffness of the isolator changes by more than plus or minus 10 percent when the cycling rate is varied from the effective frequency ($1/T_I$) at the design displacement to any frequency within the range of 0.1 to 2.0 times the effective frequency.

C161.2.5 ***Units Dependent on Bilateral Load.*** If the effective stiffness and damping of any of the isolators in the isolation system are dependent on the magnitude of the imposed bilateral (orthogonal) displacement, additional testing is required to quantify this dependence.

Reduced-scale isolators may be used to capture this dependence, provided that the reduced-scale isolators are fabricated using the same processes and quality control procedures as the full-size isolators. Dimensional relationships between full and reduced scale units should be established and verified prior to finalizing the testing program.

The criteria to be used to judge whether the properties of an isolation system are dependent on bilateral loading are specified; namely, the isolators are to be considered to be dependent on bilateral displacement if the effective stiffness at 100 percent bilateral displacement differs from the effective stiffness at 0 percent bilateral displacement by more than plus or minus 10 percent.

C161.2.6 ***Downward Vertical Load.*** The static vertical load test is used to verify isolator stability at the total maximum displacement under maximum and minimum vertical loads. The maximum vertical load is calculated using 1.2 DL + 1.0 LL and the downward seismic overturning load from the MCE. The minimum vertical load is calculated using 0.8 DL and maximum upward seismic overturning load from the MCE.

C161.2.7 No Commentary provided.

C161.2.8 ***Testing Similar Units.*** Prototype tests are not required if the isolator unit is of similar size and internal geometry, of the same type and material, and constructed using the same processes as a prototype isolator unit that has been previously tested using the specified sequence of tests. The independent engineering team should determine whether the results of previously tested units are suitable, sufficient, and acceptable.

C161.3 Determination of Force-Deflection Characteristics

The force-deflection characteristics of the isolation system shall be based on the results of cyclic testing of a selected sample of isolators.

The effective stiffness of an isolator shall be calculated for each cycle of loading from displacement Δ^+ to Δ^- (where $|\Delta^+| = |\Delta^-| = \Delta$) using Equation 161-3:

$$k_{eff} = \frac{F^+ - F^-}{\Delta^+ - \Delta^-}$$

where F^+ is the lateral force in the isolator at Δ^+; and F^- is the lateral force in the isolator at Δ^-, assuming maximum force to occur at maximum displacement.

C161.4 No Commentary provided.

C161.5 Design Properties of the Isolation System

C161.5.1 ***Effective Stiffness.*** Effective stiffness of the isolation system is determined from the force-displacement (hysteresis) loops for individual isolators as shown in Figure C151-2. The values of maximum effective stiffness and minimum effective stiffness can be calculated as noted in this figure.

C161.5.2 ***Effective Damping.*** The effective damping (β) given by Equation 161-4 is used to characterize the energy dissipation afforded by the isolation system. The maximum effective stiffness of the isolation system is used to provide a lower bound (conservative) estimate of the effective damping.

References

ACI-318, 1995. American Concrete Institute, *Building Code Regulations for Reinforced Concrete*, ACI-318-95.

ACI, 1983. American Concrete Institute, *Recommended Practice for the Design and Construction of Concrete Bins, Silos and Bunkers for Storage of Granular Materials*, ACI-313R-1977 (Revised 1983).

ACI, 1985. American Concrete Institute, *Code Requirements for Nuclear Safety-Related Concrete Structures*. ACI-349-85/349R-85.

ACI, 1989. American Concrete Institute, *Concrete Environmental Engineering Structures*. ACI 35OR-89.

API, 1988. American Petroleum Institute, API Standard 650, *Welded Steel Tanks for Oil Storage*, 8th Edition.

ATC 3-06, 1987. "ATC 3-06 Tentative Provisions for the Development of Seismic Regulations for Buildings." NBS SP-510, June 1978. "NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings." FEMA 95, February 1986.

ATC-7, 1981. *Guidelines for the Design of Horizontal Wood Diaphragms.* Applied Technology Council, 555 Twin Dolphin Drive, Redwood City, California, 94065.

AWWA, 1984. American Water Works Association, *Welded Steel Tanks for Water Storage*, ANSI/AWWA D100-84.

AWWA, 1986. American Water Works Association, *Wire-Wound Circular Prestressed Concrete Tanks*, AWWA D110-86.

Bertero, V. V., 1986. "Lessons Learned from Recent Earthquakes and Research and Implications for Earthquake-Resistant Design of Building Structures in the United States," *Earthquake Spectra*, Vol. 2, no. 4. Earthquake Engineering Research Institute, Oakland, California.

Bertero, V. V., F.M. Bendimerad, and H.C. Shah, 1988. *Fundamental Period of Reinforced Concrete Moment Resisting Frame Structures,* The John A. Blume Earthquake Engineering Center Report No. 87, Stanford University, Stanford, California.

Biggs, J. M., 1964. *Introduction to Structural Dynamics,* McGraw-Hill Book Company.

Bouwkamp, R.O. Hamburger, Gillengerten, 1991. *Degradation of Plywood Roof Diaphragms Under Multiple Earthquake Loading,* SMIP 90, May, CDMG Strong Motion Instrumentation Program, Sacramento, California.

CDMG-SMIP records, OSMS 84-7 and later. CDMG Strong Motion Instrumentation Program, Sacramento, California.

Celebi, H., G. Bongiovanni, E. Safak, and A.G. Brady, 1989. "Seismic Response of Large-Span Roof Diaphragms," *Earthquake Spectra,* Vol. 5, no. 2, Earthquake Engineering Research Institute, Oakland, California.

Chopra, A. K., 1980. *Dynamics of Structures, A Primer*, Earthquake Engineering Research Institute, Oakland, California.

Clough, R. W., and J. Penzien, 1975. *Dynamics of Structures*, McGraw-Hill, New York.

Der Kiureghian, A., 1980. "Structural Response to Stationary Excitation," *Journal of the Engineering Mechanics Division*, ASCE, Vol. 106, No. EMO.

Der Kiureghian, A., 1981. "A Response Spectrum Method for Random Vibration Analysis of MDF Systems," *Earthquake Engineering and Structural Dynamics*, Vol. 9, John Wiley & Sons.

Dobry, R. and M. Vucetic, 1987. "Dynamic Properties and Response of Soft Clay Deposits" State-of-the-Art Paper, International Symposium on Geotechnical Engineering of Soft Soils, Mexico City, August 13-14.

Kan, C. L., and A.K. Chopra, 1977. "Elastic Earthquake Analysis of Torsionally Coupled Multi-Story Buildings," *Earthquake Engineering and Structural Dynamics*, Vol. 5, John Wiley & Sons.

Kelley, J., 1993. *Earthquake-Resistant Design in Rubber*, Springer-Verlag.

Liu, S.C., L.W. Fogel, and M.R. Dougherty, 1977. "Earthquake-Induced In-Building Motion Criteria," *Structural Journal, ASCE*, January.

NEHRP, 1994. *Recommended Provisions for the Development of Seismic Regulations for New Buildings*, Federal Emergency Management Agency, Washington D.C.

OES, 1989. *Guidance for the Preparation of a Risk Management and Prevention Program*, State of California Office of Emergency Services, Hazardous Materials Division, Sacramento, California.

SEAOSC, 1977. *Seismic Analysis by Computer.* Electronic Computation Committee of the Structural Engineers Association of Southern California, 1977, SEAOSC, 2550 Beverly Blvd., Los Angeles, California, 90057.

Seed, H. B., and I.M. Idriss, 1982. *Ground Motions and Liquefaction During Earthquakes*, Earthquake Engineering Research Institute, Oakland, California.

Singh, J. P., 1988. "Strong Ground Motion and Geotechnical Aspects of Mexico, Chile, and El Salvador Earthquakes — A Review," Proceedings of EERI Seminar entitled *Evolving Earthquake Hazard Mitigation Practices, Implications from the Recent Chile, Mexico, and El Salvador Earthquakes*, Mesa, Arizona, February 4, 1988.

Stratta, J. L., 1987. *Manual of Seismic Design,* Prentice-Hall, Inc., Englewood Cliffs, New Jersey.

Tena-Colunga, 1992. "Seismic Evaluation of Unreinforced Masonry Structures with Flexible Diaphragms," *Earthquake Spectra,* Earthquake Engineering Research Institute, Vol. 8, no. 2, Oakland, California.

U.S. AEC, 1963. *Nuclear Reactors and Earthquakes*, U.S. Atomic Energy Commission, TID 7094.

U.S. Army, 1986. *Seismic Design Guidelines for Essential Buildings,* Technical Manual, Army TM 5-809-10-1.

U.S. Army, 1992. *Seismic Design for Buildings,* Technical Manual: Army TM 5-809-10; Navy NAVFAC P-355; Air Force AFM 88-3, Chapter 13.

U.S. Navy, 1982. *Foundations and Earth Structures,* Design Manual, Chapter 5. Navy NAVFAC DM-7.2.

U.S. NRC, 1976. Regulatory Guide 1.92, *Combining Modal Responses and Spatial Components in Seismic Response Analysis,* U.S. Nuclear Regulatory Commission, Revision 1.

Werner, S. D. et al., 1979. "Structural Response to Traveling Seismic Waves," *Journal of the Structural Division*, ASCE, Vol. 105, No. ST 12.

Wilson, E. L. and A. Habibullah, 1987. "A Static and Dynamic Analysis of Multistory Buildings Including *P*-Delta Effects," *Earthquake Spectra, Journal of the EERI*, Vol. 3, no. 2.

Wilson, E. L., et al., 1981. "A Replacement for the SRSS Method in Seismic Analysis," *Earthquake Engineering and Structural Dynamics*, Vol. 9, John Wiley & Sons.

Wilson, E.L. and M.R. Button, 1982. "Three-Dimensional Dynamic Analysis for Multi-Component Earthquake Spectra," *Earthquake Engineering and Structural Dynamics*, Vol. 10, John Wiley & Sons.

Winters, C.W. and M.C. Constantinou, 1993. *Evaluation of Static Response Spectrum Analysis Procedures of SEAOC/UBC for Seismic Isolated Structures.* NCEER-93-0004, State University of New York at Buffalo, NY.

Zsutty, T. C., and H.C. Shah, 1981. "The Purpose and Effects of Earthquake Codes," *Bulletin of the Seismological Society of America*, El Cerrito, California.

Table C154-1. *Damping Coefficients*

Effective Damping, β (%)	*B*	Velocity-Domain Spectra Ratio[1]
≤ 2	0.8	≤ 0.81
5	1.0	1.00
10	1.2	1.20
20	1.5	1.53
30	1.7	1.80
40	1.9	2.07
≥ 50	2.0	≥ 2.34

1. Based on the median spectrum amplification factors, velocity domain, Table 2 of Newmark and Hall, 1982

Table C155-1. *Limits on Key Design Parameters*

		Dynamic Analysis	
Design Parameter	**Static Analysis**	**Response Spectrum**	**Time History**
Design Displacement	D=10 ZNST/B	—	—
Total Design Displacement	$D_T \geq 1.1\ D$	$\geq 0.9\ D_T$	$\geq 0.9\ D_T$
Total Maximum Displacement	$D_{TM}=M_M D_T$	$\geq 0.8\ D_{TM}$	$\geq 0.8\ D_{TM}$
Design Shear (at or below isolation system)	$V_b=K_{max}D/1.5$	$\geq 0.9\ V_b$	$\geq 0.9\ V_b$
Design Shear ("regular" superstructure)	$V_s=k_{max}D/R_{wl}$	$\geq 0.8\ V_s$	$\geq 0.6\ V_s$
Design Shear ("irregular" superstructure)	$V_s=k_{max}D/R_{wl}$	$\geq 1.0\ V_s$	$\geq 0.8\ V_s$
Drift	$0.010/R_{wl}$	$0.015/R_{wl}i$	$0.020/R_{wl}$

A_a = EPA in g's

A_v = EPV/30 in g's; EPV is in inches/seconds

A_v = velocity related acceleration value

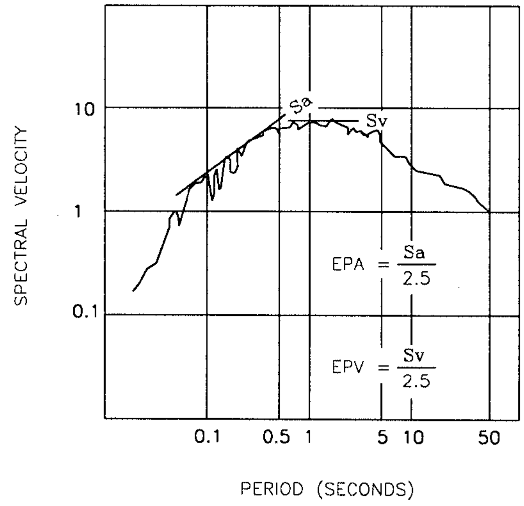


Figure C104-1. *Schematic representation showing how effective peak acceleration and effective peak velocity are obtained from a response spectrum*

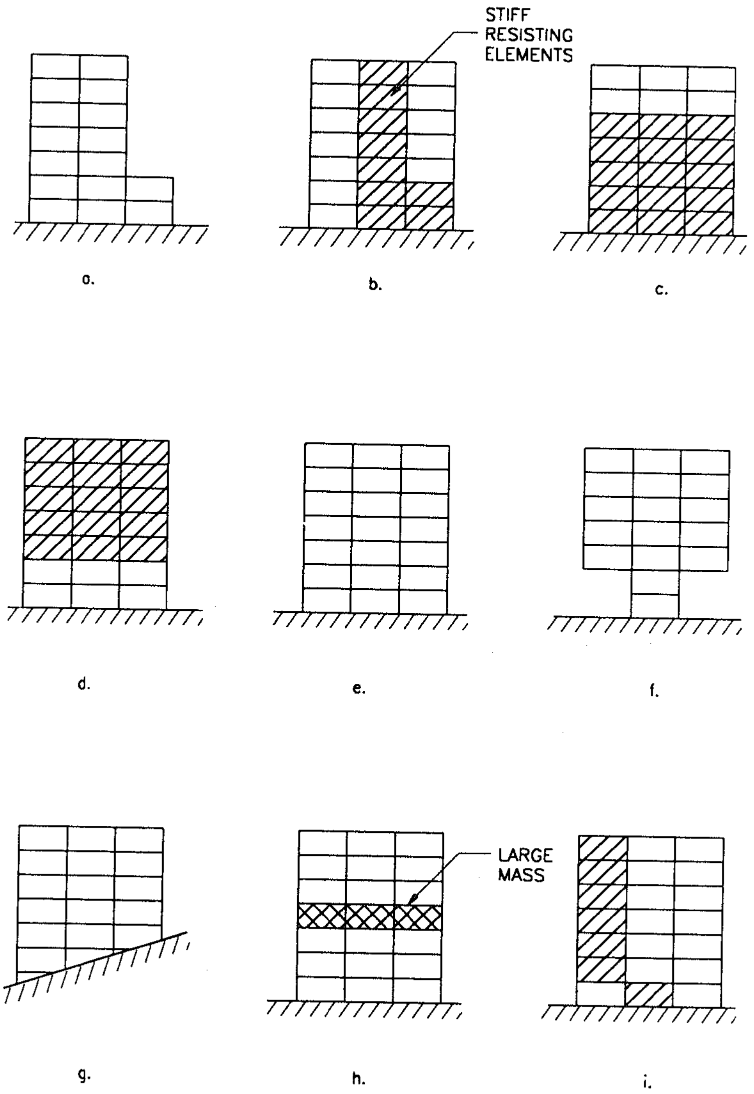


Figure C104-2. *Irregularities in elevation*

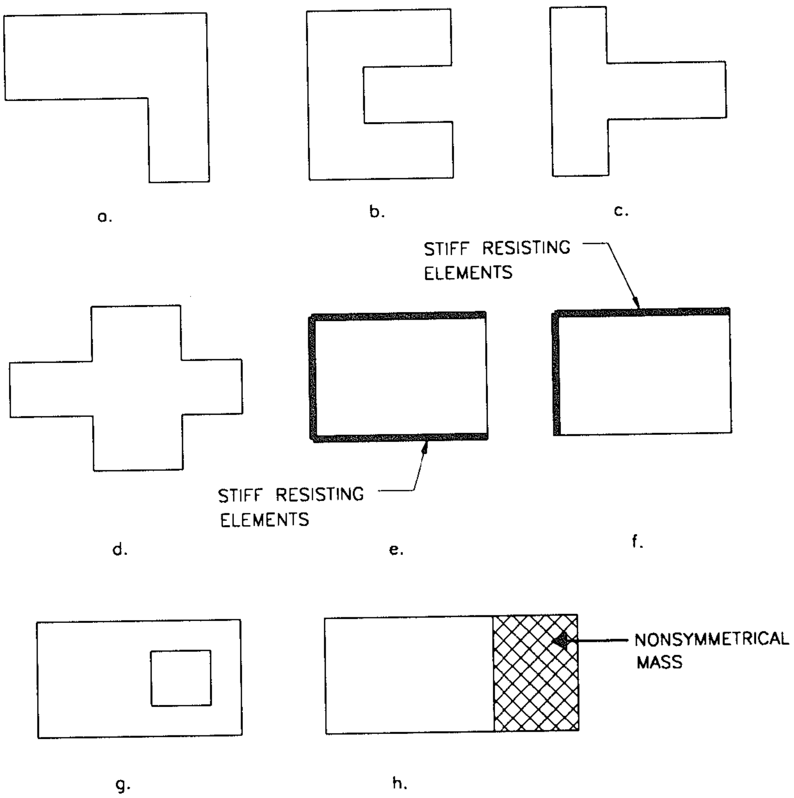


Figure C104-3. *Irregularities in plan*

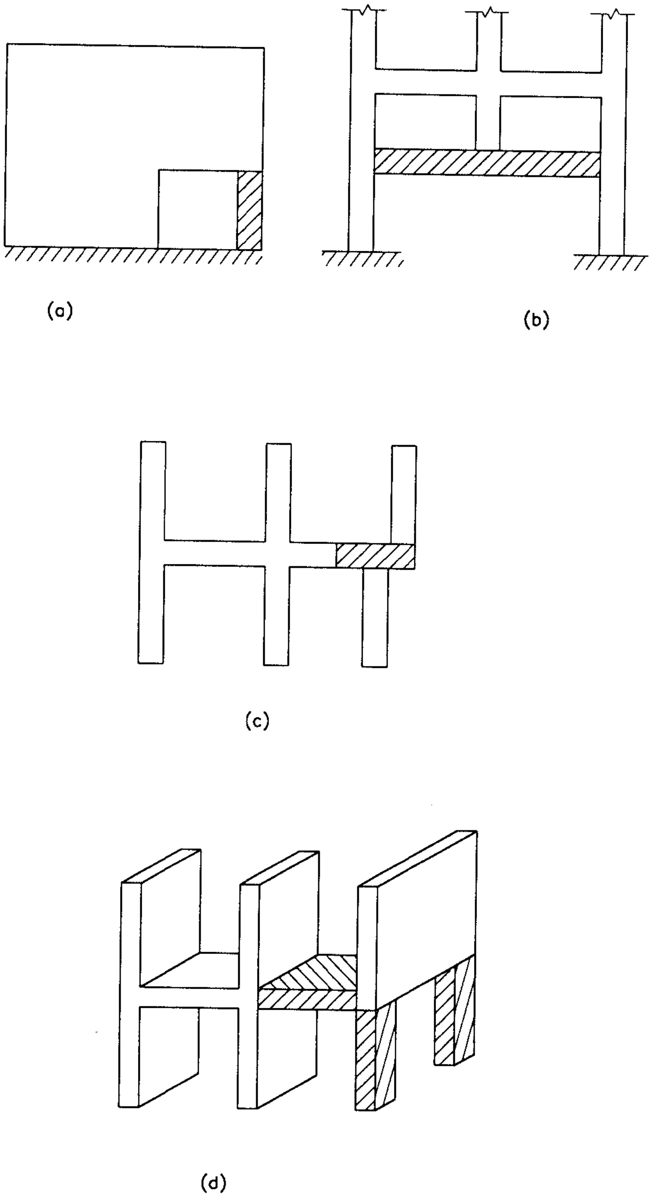
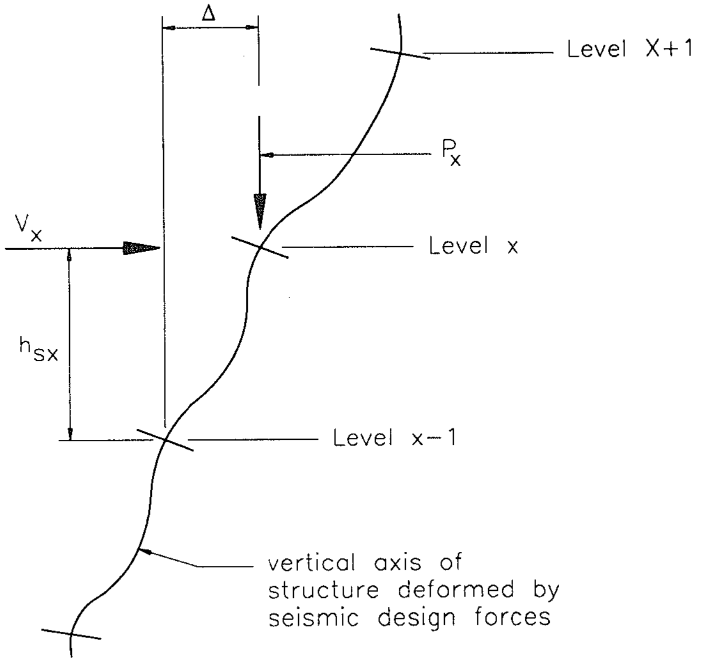


Figure C104-4. *Irregularities in force transfer*


Δ
Level X+1
P_x
V_x
Level x
h_{sx}
Level x−1
vertical axis of structure deformed by seismic design forces

Δ = design story drift

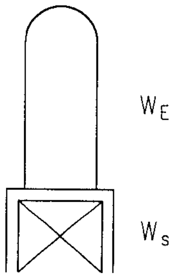
V_x = seismic design shear force acting between level x and level x-1

h_{sx} = story height below level x

$P_x = \sum_{i=x}^{n} (D+L)i$ total design gravity load and above level x

Stability coefficient: $\Theta = \dfrac{P_x \Delta}{V_x \bullet h_{sx}}$

Figure C105-1. *P-delta symbols and notations*



W_E less than 0.25 W_s
T_E greater than 0.06 sec.
(flexible)

Support Structure

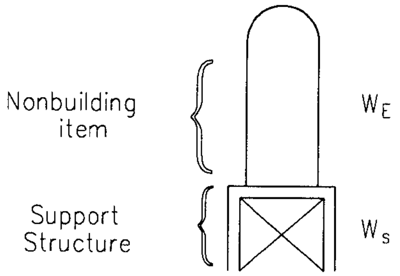
$V=\frac{ZICW}{R_w}$ with R_w from Table 104–6 or 104–8

(lump mass of item with mass of structure)

Nonbuilding Item and Achorage

Use larger of $V = \frac{ZICW}{R_w}$ from Table 104–7

or $V=Z(2.0 \times Cp)\ IpW$ with Cp from Table 104–8



W_E less than 0.25 W_s
T_E less than 0.06 sec.
(rigid)

Support Structure

$V = \frac{ZICW}{R_w}$ with R_w from Table 104–6 or 104–8

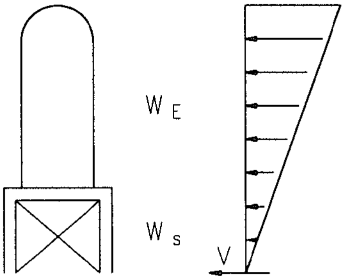
(lump mass of item with mass of structure)

Nonbuilding Item and Anchorage

Use larger of $V = 0.5\ ZIW$

or $V=ZCp\ IpW$ with Cp from Table 104–7

Figure C109-1. *Nonbuilding structures supported above grade ($W_E < 0.25 W_s$)*



W_E greater than 0.25 W_s
T_E greater than 0.06 sec.
(flexible)

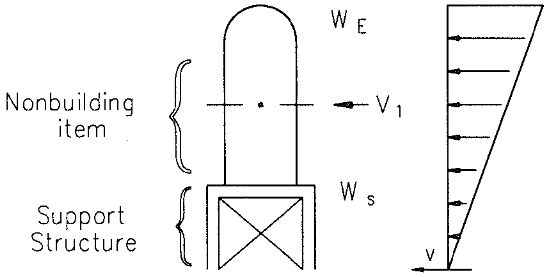
Support Structure

$V = \frac{ZICW}{R_w}$ with R_w from the smaller of Table 104-6 and Table 104-8

with $\frac{C}{R_w} > 0.5$

Cmax = 2.75

C based on combined system T



W_E greater than 0.25 W_s
T_E less than 0.06 sec.
(rigid)

Support Structure

$V = \frac{ZICW}{R_w}$ with R_w from Table 104-6 or 104-8
(lump mass of item with mass of structure)

Nonbuilding Item and Anchorage

Use larger of $F_E = 0.5\ ZIW_E$

or $F_E = ZCp\ IpW_E$ with Cp from Table 104-7

Figure C109-2. Nonbuilding structures supported above grade ($W_E < 0.25W_s$)

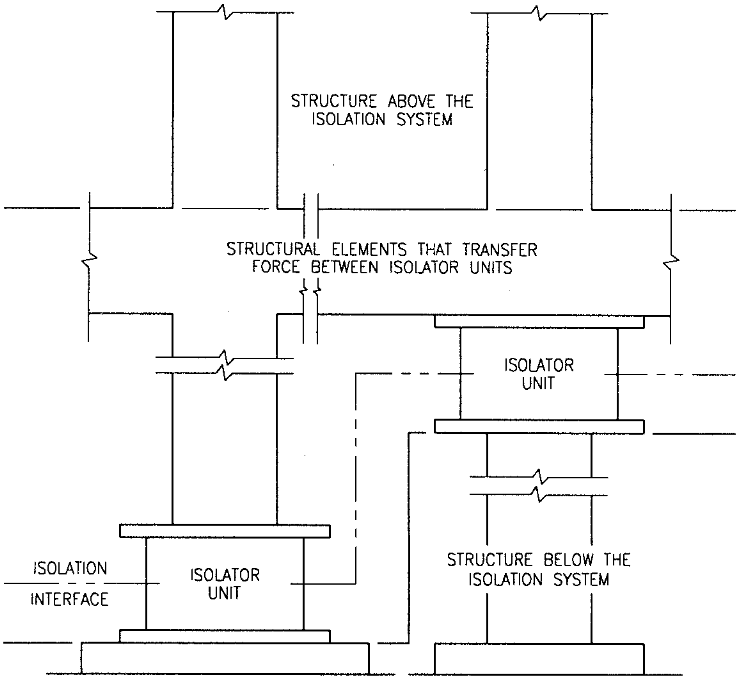
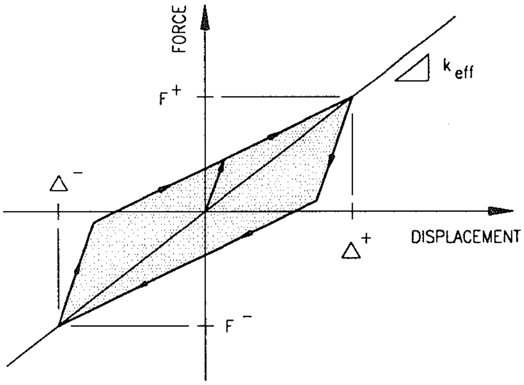
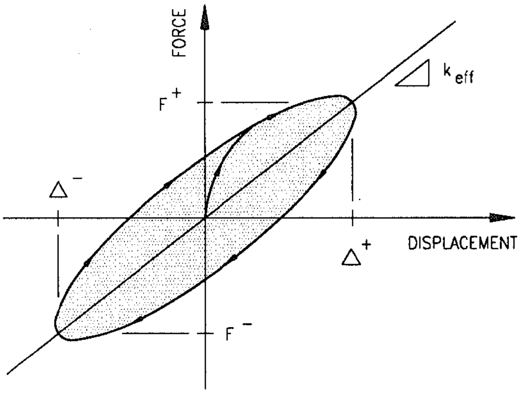

STRUCTURE ABOVE THE ISOLATION SYSTEM
STRUCTURAL ELEMENTS THAT TRANSFER FORCE BETWEEN ISOLATOR UNITS
ISOLATOR UNIT
ISOLATION INTERFACE
ISOLATOR UNIT
STRUCTURE BELOW THE ISOLATION SYSTEM

Figure C151-1. *Isolation system terminology*



HYSTERETIC SYSTEM



VISCOUS SYSTEM

Figure C151-2. *Idealized isolation system force-displacement relationships*

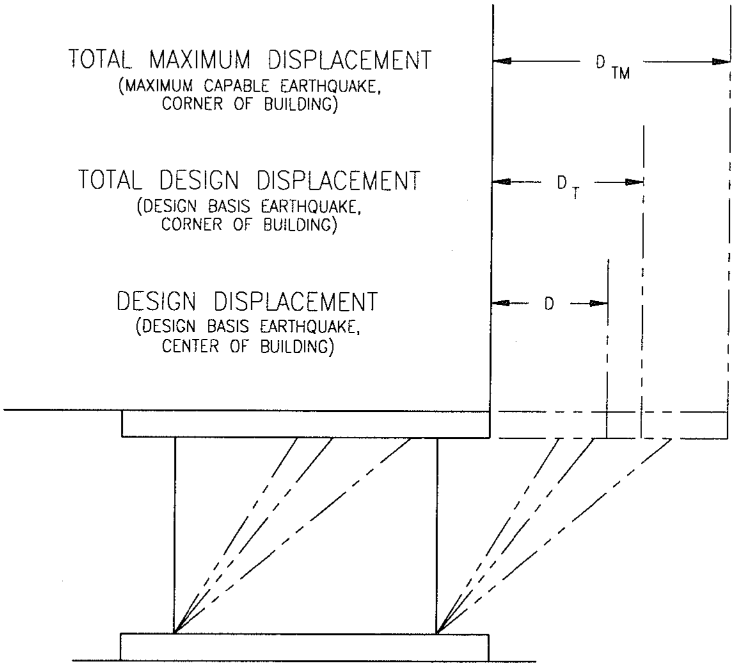

TOTAL MAXIMUM DISPLACEMENT
(MAXIMUM CAPABLE EARTHQUAKE, CORNER OF BUILDING)
D_{TM}
TOTAL DESIGN DISPLACEMENT
(DESIGN BASIS EARTHQUAKE, CORNER OF BUILDING)
D_T
DESIGN DISPLACEMENT
(DESIGN BASIS EARTHQUAKE, CENTER OF BUILDING)
D

Figure C151-3. *Displacement terminology*

APPENDIX 1A

Seismic Zone Coefficient

This appendix provides background and commentary for the Z coefficients, as presented in the 1994 UBC, and was prepared in the late 1980s. More recent earthquakes and related research have resulted in further development of ground motions currently being proposed by SEAOC for the 1997 UBC. Near source effects have been particularly addressed and the reader is referred to Appendix A, which describes some of these effects experienced during the 1994 Northridge earthquake.

The Z coefficient, or seismic zone coefficient in Equation 104-1 (see Requirements) for the design base shear takes into account geographical variations in the expected levels of earthquake ground shaking. Evidence for such variations can be found both in the historical record of earthquake effects, and also in the distribution of the major fault systems considered likely to produce significant earthquakes.

The zone map for California is shown in Figure 104-1 (see Requirements). This map is based primarily on historical seismicity and was derived from the Applied Technology Council (ATC) zoning maps [ATC 3-06, 1978], which are based on probabilistic maps by Algermissen and Perkins [1976]. These probabilistic maps estimated peak horizontal acceleration on rock that has a 10 percent probability of being exceeded in a 50-year period. The ATC maps estimate effective peak acceleration (EPA) on rock, which controls the short period (constant acceleration) design base shear, and effective peak velocity (EPV), which controls the longer period base shear (constant velocity on a tripartite spectrum).

The zone map (Figure 104-1) used in these Requirements presents only acceleration values. Both the EPA and the acceleration related to EPV were compared; the higher value was used to determine the map boundaries. The Z values for Zones 3 and 4, respectively, are 0.3 and 0.4 for use in the design base shear formula, which was modified so that values of Z would correspond to the estimated values of effective peak acceleration. It was intended that the boundary between Zone 3 and Zone 4 correspond to an effective peak acceleration of 0.3g (or equivalent velocity).

The map is based principally on the ATC maps with modifications reflecting other sources of information, including the maps of acceleration and velocity published by Algermissen et al. [1982]. The ATC maps and the maps by Algermissen et al. are based primarily on the historical record of earthquakes, which in California is short relative to the repeat time of major earthquakes, even on the most active faults. To avoid the danger of missing areas that are potentially active, but simply by chance have not shown activity during the short time of the historical record, geologic data were consulted, particularly data on fault slip rates.

The importance of considering geologic data in evaluating earthquake potential was demonstrated by Allen [1975]. On the basis of geologic data, Zones 3 and 4 were extended in southeastern California to encompass the Garlock fault. This modification was made to be consistent with the criterion that the Z coefficient correspond to ground motion values with a 10 percent probability of being exceeded in 50 years.

Ground motion in Zone 4 near major faults requires special attention, particularly for structures with periods greater than about 0.3 second. The authors of the ATC 3-06 Commentary [ATC 3-06, 1978] acknowledged the possibility of near-fault ground motion higher than that corresponding to $Z=0.4$. Since the time the ATC-3 Commentary was written, data from the Imperial Valley earthquake of 1979 confirm the existence of a narrow zone of high ground motion near the causative fault. The problem is illustrated in Figure App1A-1, which shows the horizontal pseudoacceleration response at 1.0 second with 5 percent damping plotted against distance from the fault for strong-motion sites in the 1979 Imperial Valley earthquake. The response values plotted are directly comparable to the product ZC in the formulas of the equivalent static lateral force procedure. The one-second ZC values for Zone 4 with site conditions S_2 and S_3 are shown in Figure App1A-1 for comparison. (All the points plotted in Figure App1A-1 correspond to site conditions S_2 or S_3). The important thing to note is the existence of a zone within 5 miles of the fault in which the one-second response increases by a factor of 2.

Figure App1A-1 also shows that the one-second response in the 1979 Imperial Valley earthquake (Richter local magnitude 6.6) did not greatly exceed the Zone 4 ZC values, which is reassuring, but the response may be substantially greater in earthquakes of larger magnitude. Neither strong motion data nor historical data on performance of structures are adequate for empirical determination of design ground motion values close to an earthquake of magnitude greater than 7.0. Recent estimates by different authors disagree; generally they show an increase with magnitude, but differ as to the amount [Allen, 1995; Campbell, 1988; Donovan and Bornstein, 1978; Idriss, 1987; Joyner and Boore, 1981; Joyner and Boore, 1982; McGuire, 1974). The increase with magnitude is larger for long-period motions than for short-period motions. Figure App1A-2 shows the horizontal pseudoacceleration response at 1.0 second with 5 percent damping for a magnitude 7.5 earthquake at a soil site, as given by four widely used prediction equations by different authors. The ZC value for Zone 4 and site condition S_2 is shown for comparison. The different equations give widely divergent values at short distance, reflecting the lack of strong motion data, but all four give values substantially in excess of the Zone 4 ZC value.

Jurisdictions close to major faults may wish to have special zoning studies made, as suggested in the ATC-3 Commentary [ATC 3-06, 1978). A structure well designed in accordance with these Requirements for Zone 4 may have sufficient margins of safety to accommodate the larger motions that may occur near the major faults. To ensure that the full margins of safety are realized, however, SEAOC urges that at sites within 5 miles of the San Andreas, San Jacinto, Imperial, and Garlock faults, the engineer and building official exercise special care that the provisions of these Requirements and of good engineering practice are fully met, particularly for structures with periods greater than 0.3 second. Distances to the listed faults can readily be determined using the Alquist-Priolo maps published by the California Division of Mines and Geology at a scale of 1:24,000 (about 2.6 inches to the mile).

References

Algermissen, S.T., D.M. Perkins, P. C. Thenhaus, S. L. Hansen, and B.L. Bender, 1982. *Probabilistic Estimates of Maximum Acceleration and Velocity in Rock in the Contiguous United States,* U.S. Geological Survey Open-File Report 82-1033.

Algermissen, S.T., and D.M. Perkins, 1976. *A Probabilistic Estimate of Maximum Acceleration in Rock in the Contiguous United States,* U.S. Geological Survey Open-File Report 76-416.

Allen, C.R., 1975. "Geological Criteria for Evaluating Seismicity, " *Bulletin of the Geological Society of America*, Vol. 86.

ATC 3-06, 1978. Tentative Provisions for the Development of Seismic Regulations for Buildings. ATC 3-06, Applied Technology Council, Redwood City, California.

Brady, A.G., V. Perez, and P.N. Mork, 1980. *The Imperial Valley earthquake, October 15, 1979: Digitization and Processing of Accelerograph Records,* U.S. Geological Survey Open File Report 80-703.

Campbell, K. W. and K.W. Campbell, 1988. "Predicting Strong Ground Motion in Utah, " *Evaluation of Regional and Urban Earthquake Hazards in Utah,* W.W. Hays and P. L. Gori, Editors, USGS Professional Paper.

Donovan, N. C. and A. E. Bornstein, 1978. "Uncertainties in Seismic Risk Procedures, " *Proceedings of the American Society of Civil Engineers, J. Geotech. Eng. Div.,* Vol. 104, no. GT7.

Idriss, I. M., 1987. "Earthquake Ground Motions," Lecture Notes, Strong Ground Motion Seminar, Pasadena, California, April 10-11. Earthquake Engineering Research Institute, Oakland, California.

Joyner, W.B., and D.M. Boore, 1982. "Prediction of Earthquake Response Spectra," *Proceedings of the 51st Annual Convention*, Structural Engineers Association of California, Sacramento, California.

Joyner, W.B., and D.M. Boore, 1981. "Peak Horizontal Acceleration and Velocity From Strong-Motion Records Including Records From the 1979 Imperial Valley, California, Earthquake," *Bulletin of the Seismological Society of America*, Vol. 71, El Cerrito, California.

McGuire, R.K., 1974. "Seismic Structural Response Risk Analysis, Incorporating Peak Response Regressions on Earthquake Magnitude and Distance," Research Report R74-51, Dept. Civil Engineering, Massachusetts Institute of Technology.

Schnabel, P.B., and H.B. Seed, 1973. "Accelerations in Rock for Earthquakes in the Western United States," *Bulletin of the Seismological Society of America*, Vol. 63, El Cerrito, California.

Trifunac, M.D., and J.G. Anderson, 1978. "Preliminary Empirical Models for Scaling Pseudo-Relative Velocity Spectra," Appendix A in *Methods for Prediction of Strong Earthquake Ground Motion*, U.S. Nuclear Regulatory Commission NUREG/CR-0689.

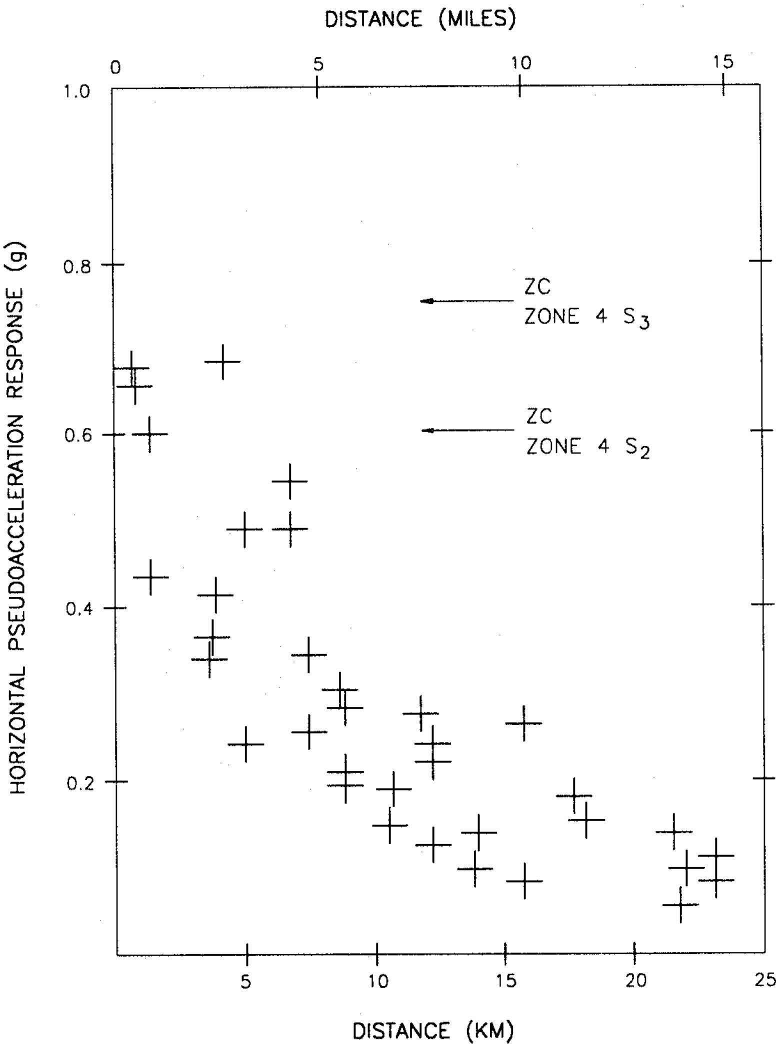


Figure App1A-1. *Horizontal pseudoacceleration response (g) for 1.0 second period and 5 percent damping recorded at soil sites in the 1979 Imperial Valley earthquake [Trifunac and Anderson, 1978]. The values of Zc in Zone 4 for soil conditions S_2 and S_3 are shown for comparison.*

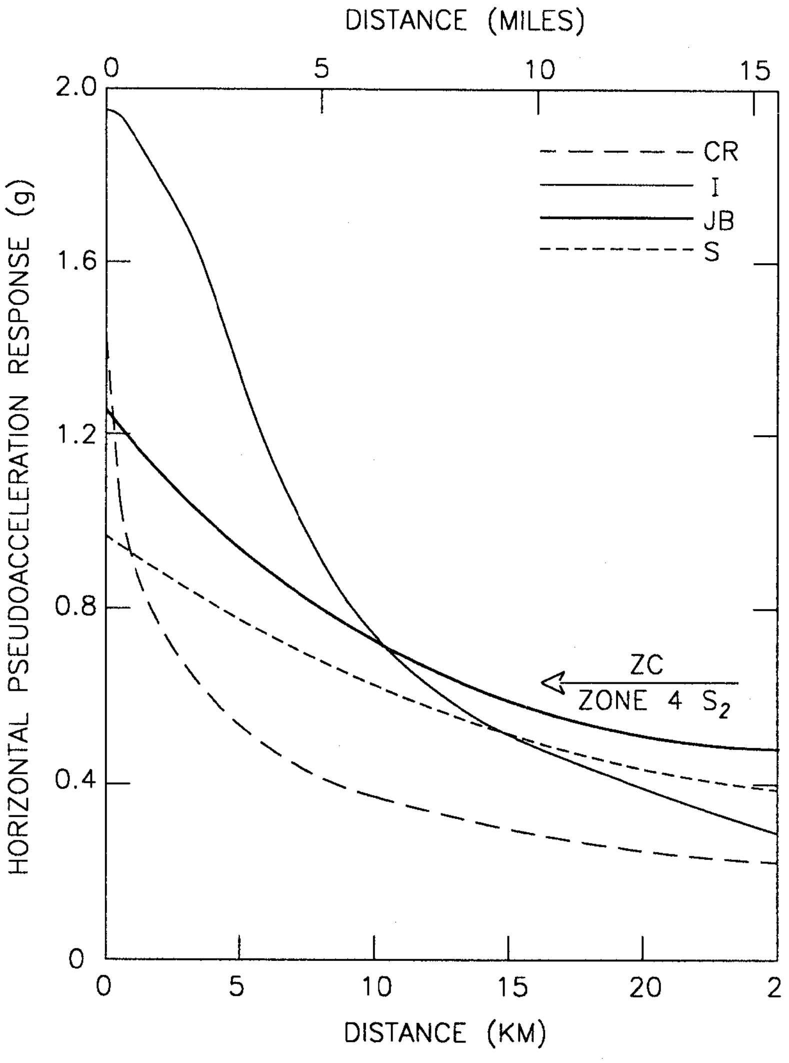


Figure App1A-2. *Horizontal pseudoacceleration response (g) for 1.0 second periods and 5 percent damping for a magnitude 7.5 earthquake at a soil site according to four widely used prediction equations. CR, from Crouse [1987]; I, from Idriss [1987]; JB, from Joyner and Boore [1982]; S, from Sadigh [1987]. The value of ZC in Zone 4 for soil condition S_2 is shown for comparison.*

APPENDIX 1B
C and S Coefficients

This appendix provides background and commentary for the C and S coefficients, as presented in the 1994 UBC, and was prepared in the late 1980s. More recent earthquakes and related research have resulted in further development of ground motions currently being proposed by SEAOC for the 1997 UBC. The reader is referred to the 1994 NEHRP Provisions and Commentary for an updated discussion of site effects and soil factors.

A simple method to account for variable seismic demand as a function of site soil conditions is necessary for use in the equivalent lateral force method. In Equation 105-2 (see Requirements), a site coefficient related to the site soil dynamic characteristics is used. Values are set forth in Table 104-2 (see Requirements).

Softer soils both amplify the response of the site to ground motion and increase the width of the spectrum (see Soil Types 2 and 3 on Figure 104-2 in Requirements Chapter 1). Use of the simple approach should be avoided for very soft soils (Type S-4), such as bay muds. A site specific soils investigation should be considered where Type S-4 conditions are encountered.

The need for as simple an expression of the complete lateral force coefficient as possible has already been described. This simplicity is achieved with the recommended soil coefficients by using the values given in Table 104-2 (see Requirements) with the added restriction that at short response periods the value of C, which also includes the site coefficient S and is computed by using Equation 105-2 (see Requirements), need not exceed a value of 2.75.

A comparison of the spectral ratios for the three spectra shown in Figure 104-2 (see Requirements), with the S factor values given in Table 104-2 (also in Requirements), will show that those portions of the spectra that are different for each soil profile type do not have the same ratios. This difference arises from the relationship, which uses the lateral force coefficient as a single design value for all types of structures. In a design analysis that uses a response spectrum, the effects of individual modes are computed separately and then combined to give the total building response. The more complex buildings are usually taller, and as a consequence have longer fundamental response periods. A simple method of both representing the combined mode effect and of introducing a more *conservative* design approach for long period structures is achieved by reducing the exponent, which is used with the period of the structure T

in the formulation of the factor C. The reduction of the exponent from a value of 1 to a value of $^2/_3$ results in a slower attenuation of the lateral force coefficient with increasing period than is obtained with the corresponding response spectrum.

To obtain a balance between the overall curves when the different exponent was used, it became necessary to modify the values of the site coefficients in Table 104-2. Each factor in Table 104-2 is effective over a different and increasingly longer period range. Similarly the slower attenuation effect increases with increasing period. The results of this trade-off is that, whereas the values in Table 104-2 range from 1.0 to 1.5 for the first three soil types, the ratios of the separate spectral values range between 1.0 and 2.0 for the same three soil types. A similar comparison for Soil Type 4 cannot be made because of the wide variety of response spectra that could be produced by soils in this group.

A study performed by the Structural Engineers Association of Northern California (SEAONC) Site Effects Committee in 1982 showed that the values of lateral force coefficients determined by the 1974 S-factor approach and the ATC 3-06 S-factor approach ($S1$, $S2$, and $S3$) gave generally similar results for a range of building types and soil conditions. Thus, it was considered appropriate to change from the 1974 approach to the somewhat simpler ATC 3-06 approach. At the same time, reflecting studies of site effects in Mexico City in 1985, an extra site condition ($S4$) has been added to the original three site conditions.

It is strongly recommended that for structures located on soft soil profiles, a site-specific study should be performed to develop seismic design criteria. The value of the site coefficient for soils of the $S4$ classification was included for guidance purposes, in recognition of the fact that a site-specific study may not be warranted for some structures (see Sections 104.8.2.4 and 105.2.4).

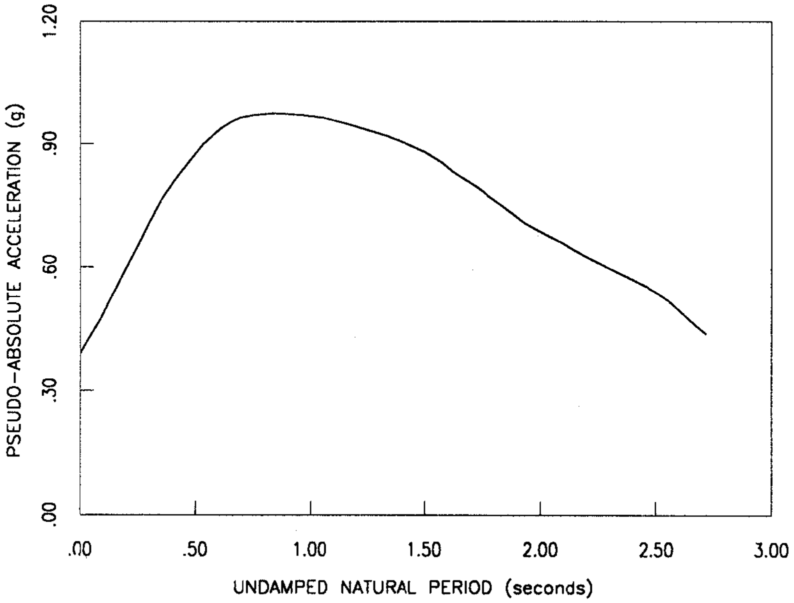


Figure App1B-1. *Representative response spectrum for soft clay site; damping=5 percent; Z=0.40*

APPENDIX 1C

Development of the R_W Factor

The term "ductility" is widely used to describe the energy dissipation capability of a structural element or system. While this term may apply to certain specific elements and systems, it is not adequate to describe the ability of most systems to sustain strong seismic ground motion without collapse. Redundancy, reliability of as-built performance, inelastic load-deformation behavior, and changed damping and period modification with deformation, all affect the capability of a structural system to resist severe earthquake ground motion without collapse. The use of "ductility" to describe this complex behavior was judged to be inappropriate and subject to misinterpretation, since ductility is widely understood to be a fundamental property of materials, and not a property of structural system response. In these Requirements, a more general terminology is employed such as: special moment resisting frames rather than "ductile frames" and structural system Quality Factor R_w rather than "system ductility." An extended discussion of the R_w factor follows.

SEAOC proceeded in a deliberate manner to assess relative R_w values. First, the types of structural framing systems were listed (as in Table 104-6 in Requirements) with an effort to include all modern, accepted methods of construction. Then, the factors or characteristics to be considered in the evaluation of R_w were determined to include:

1. Observed system performance.
2. Level of inelastic response capability.
3. Possibilities of vertical load system failure.
4. Lateral force system redundancy.
5. Multiplicity of lines of defense, such as backup frames.

The structural systems were then divided into categories within which some of these characteristics were common. These categories and their abilities to meet the listed characteristics are briefly reviewed below:

Moment Resisting Frame System

1. Good performance where energy dissipative special details are used.
2. High level of inelastic response capacity.
3. System can perform inelastically without jeopardizing vertical capacity and stability.

4. Systems in buildings that have performed well in past earthquakes have been highly redundant.

Dual System

1. Good performance.
2. Good elastic response control for moderate shaking—good energy dissipation for strong shaking.
3. Damage to primary lateral force resisting system does not affect stability of the vertical system.
4. Backup system provided in the form of a moment frame.
5. Two lines of defense.

Building Frame System

1. Performance varies depending on material and configuration.
2. Level of inelastic response is dependent on type of bracing system employed.
3. Damage to the lateral system, by definition, should not lead to failure of vertical system.
4. Redundancy not specified in description of frame.
5. Nominal backup system only in the form of a space frame.

Bearing Wall and Bearing Braced Frame System

1. Poor performance of some materials and highly variable performance, depending on the quality of construction execution.
2. Lower level of inelastic response capability.
3. Lateral system failure could lead to vertical system failure.

Once the above characteristics were established, certain agreed-upon reference structures were selected. The systems having the highest and lowest expected levels of performance are a steel ductile frame and a box type masonry or concrete building, respectively. The R_w values for these two systems were determined from the K factor of previous editions (using $R_w = 8/K$), giving R_w values of 12 and 6, respectively. No compelling arguments were offered to change the design basis loads for these systems or to change their interrelationship. The expected performance of other systems was then evaluated relative to these reference systems in order to determine the intermediate R_w values. The same characteristics discussed above were used, and considerations focused on the following issues:

1. The R_w value depends on the degree to which the system can be allowed to go beyond the elastic range, its degree of energy dissipation in so doing, and the stability of the vertical load-carrying system during inelastic response due to maximum expected ground motion. The detailing necessary to achieve this inelastic performance is covered in the design requirements and material chapters. Also, regardless of the R_w value and its corresponding system, all elements of the structure must be adequately tied together to transfer loads to the lateral force resisting elements.

2. The consequences of failure or partial failure of vertical elements of the seismic force resisting system on the vertical load carrying capacity and stability of the total building system.
3. The inherent redundancy of the system that would allow some progressive inelastic excursions without overall failure. One localized failure of a part must not lead to failure of the system.
4. When dual systems are employed, important performance characteristics include the deformation compatibility of the systems and the ability of the secondary (back-up) system to maintain vertical support when the primary system suffers significant damage. The backup system, when compatible with the primary system, can serve to redistribute lateral loads when the primary system undergoes degradation and should stabilize the building in the event that the primary system is badly damaged. If they are incompatible, then the total system performance may be very unreliable and therefore unacceptable.

It is recognized that the assigned R_w values must be periodically reviewed as earthquake performance is observed and more data on material and system performance becomes available. The relationship between the load side of the design equation, including the R_w reduction and the resistance or material side, however, should not be confused. Qualities related to the seismic load modification characteristics of the structural system should be represented by the R_w value. Qualities related to the performance and reliability of individual structural elements should be represented by either the assigned allowable stresses or the capacity reduction factors on the resistance side.

When a structural system has been designed for the seismic load level established by its R_w value, there are two behavior properties that allow the structure to perform adequately under major expected ground motion:

1. **Total System Resistance Capacity.** The total resistance capacity of a structural system is influenced by several factors. There is a factor of about 1.4 between the allowable combined load effects (1 1/3 times allowable) and the yield or strength design basis for individual elements. Then, depending on the gravity load design of the structural elements, extra design provisions, and redundancy within the total system, there may be a substantial increase in the effective inelastic lateral force resistance level of the structure as compared to individual element behavior.
2. **Dynamic Response Modification.** When structures behave inelastically under dynamic loading, they exhibit increased damping and their effective period increases. The combined effect is that the response is decreased from that projected without consideration of these changes. In terms of linear system behavior, this resistance to loading is equivalent to a reduced loading of the unaltered linear system. This can be represented as a decrease in the ordinate value of the response spectrum or *ZC* curve, which represents the unaltered linear system response to maximum expected ground motion. Figure App1C-1

provides a graphical representation of system response. Demand is represented by the ZC curve taken as the 5 percent damped multi-mode acceleration response spectrum envelope.

Shear V_A is the required lateral load design level for a given system, structural period T_A, and R_w value ($V_A = V_E/R_w$). The actual linear threshold capacity of the structure is V_B, which is greater than V_A, based on the strength level of the individual structural elements, including: load factors from the working stress level; various load combinations; extra design provisions; contributions from the nonlateral force resisting system and nonstructural elements; and as-built sizes and material strengths.

Consider the inelastic response of the building as it is loaded up to the maximum base shear, V_D. Response is expected to be linear up to shear V_B. Above V_B, nonlinear effects on period and damping begin, and continue until the inelastic capacity equals demand (V_D). Step B to D recognizes that the total resistance of the entire system increases as redundant members become fully developed, that the period increases from T_A to T_D as nonlinear softening takes place, and that the damping increases. The period change results in a decreased acceleration demand from V_E to V_C. Further, since damping has increased due to enhanced effective damping in the nonlinear system, a new demand curve (ZC), is appropriate. This effectively reduces the demand from V_C to V_D. Therefore, a system designed to the specified load and design provisions of these Requirements is expected to develop both force resistance and deformation capability exceeding the demand represented by the spectrum ordinate V_D. However, it should be recognized that the corresponding displacement response spectrum ordinate is proportional to the squared value of T_D times the acceleration response value. This displacement relationship is important, since it reinforces the warning that the displacements still can be large, even though the loads are reduced by inelastic behavior.

The R_w value assigned for each system assumes that the provisions of the material chapters (Chapters 4-8) of these Requirements are met. These govern the design and detailing of the material and/or materials used in the system. This dependence is unavoidable because R_w is a measure of the capability of the material in the participating structural elements to tolerate inelastic deformations experienced during the maximum expected ground motion. Therefore, R_w is related to the ratio of material strain at maximum response to strain at the allowable stress level. It is important to recognize that the material strain values experienced during the maximum expected seismic response are substantially greater than the strain values caused by maximum expected gravity loads (as represented by the strain at the load factored value of the working gravity loads). If an element or structural system does not have adequate details (such as confinement reinforcing in concrete, or stiffeners and panel zone reinforcement for structural steel), then it can neither tolerate the expected strains nor merit the assigned R_w value where the same design basis stresses are used for seismic loads as used for gravity loads. When adequate details are *not* provided, then either the allowable stress values for seismic load resistance must be reduced or the R_w value must be reduced. Both steel (Chapter 5) and reinforced concrete (Chapter 4) have specific details and requirements to ensure that the expected seismic strain demands can be tolerated without material failure. When these Requirements are used, both steel and concrete design can benefit from use of the 1 1/3 allowable stress values (or the load-factored equivalent for concrete), for the same allowable stresses that are used for gravity load design.

While R_w is a representation of overall ductility and energy dissipation, its value is not and shall not be equated to the "ductility" of an element. The specific value for ductility depends strongly on the particular definition used: steel beam-column connections may exhibit ductility factors of 20 or more, reinforced confined concrete has similar values, and specially inspected and detailed masonry walls may show rotational ductility values of about 15. However, it would not be appropriate to use these as a strength design reduction factor, since the result would be design loads at about one half of our present values. Experience has generally shown the assignments for R_w values, Table 104-5, to be about right to meet the objectives of Section 104, that is, structural damage limitation at moderate shaking and stability at major ground motion levels.

The R_w value must represent this anticipated as-built performance capability. It must be recognized that ductilities achieved under controlled, laboratory test conditions may not be realized in the as-built structure. Reduced values of R_w are appropriate for materials and systems having large variability in as-built properties to provide a reliable lower bound on actual capability.

The R_w values for the four basic systems correspond to the previous K factors of 0.67, 0.80, 1.00, and 1.33 ($R_w = 8/K$). As improved design procedures, and special detailing practice evolves, such as has occurred with eccentric braced frames, special concentrically braced frames, and masonry wall frames, the basic R_w factors have been adjusted to reflect the improvements in the basic systems.

The R_w values for structural systems have typically been selected based on the characteristics of vertical elements of the gravity and lateral force resisting systems including walls, frames, and bracing systems. For some classes of structures—those with relatively rigid vertical elements of the lateral force resisting system but flexible diaphragms — recent research has demonstrated that response of the system and a significant portion of the inelastic energy dissipation occurs within the flexible diaphragm elements. Many low-rise commercial and industrial buildings, including masonry and concrete wall buildings with wood diaphragms, are of this type. Historically, such buildings have been designed using restrictive R_w values based on the past poor performance of these "box type" structures. With the introduction of the ductile masonry wall frame system, it is likely that many such buildings will be designed with ductile vertical elements, but with the same diaphragm systems that have contributed to past poor performance. Similar structures could be constructed using special moment resisting concrete frames as the vertical elements. In order to encourage the use of the improved detailing practice for the vertical elements, an adjustment in the R_w value for structures with masonry wall frames was adopted. However, to limit the inelastic demands on the flexible diaphragms, these elements must still be designed using the more restrictive values commonly used in the past. Therefore, a footnote was added to Table 104-5, limiting the value of R_w used in proportioning wood diaphragms supporting concrete or masonry walls.

Except in the case of flexible diaphragm buildings, as previously described, when systems are of irregular configuration or contain member arrangements or local details that may show distress or failure, then extra design considerations, such as the $3(R_w/8)$ load factor, or special provisions are typically required, rather than changing the overall R_w value. The material resistance or allowable stresses remain the same for all systems; the R_w factor applies only to the load side of the design equation.

Future research development and earthquake performance observations are expected to provide more reliable R_w values. In the meantime, the subjective values recommended by SEAOC are believed to meet the objectives set down in Section 104.

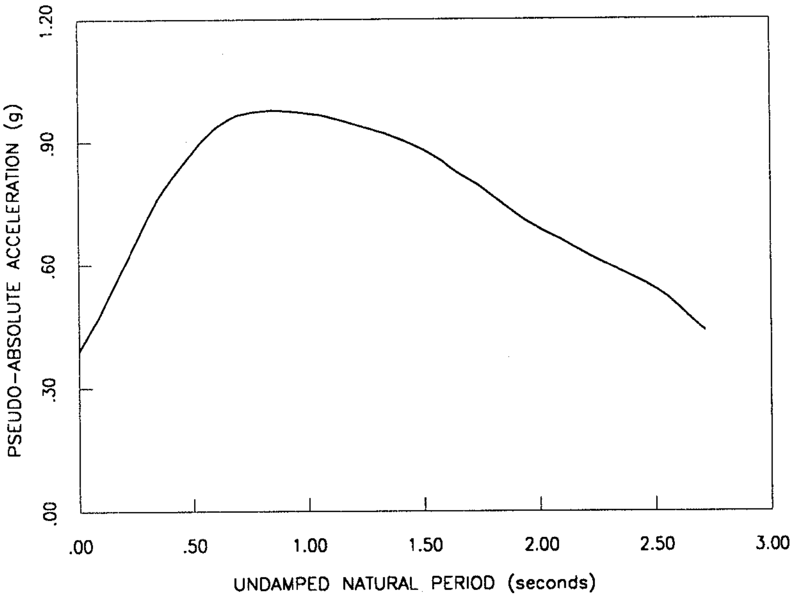


Figure App1C-1. Resistance versus demand

APPENDIX 1D
Dynamic Analysis

Multi-Dimensional Analysis Procedures

Dynamic analysis principles are set forth in many textbooks on dynamics [Wilson et al., 1981]. However, the derivations are generally only for two dimensional (planar) systems. Torsionally irregular structures, for which dynamic analysis producers are generally required per Section 104.8.3, require an understanding of multi-dimensional dynamic analysis procedures that couple torsional response with lateral response. This Appendix sets forth multi-dimensional response spectrum analysis procedures.

Symbols and Notations

k = A given direction or degree of freedom of motion of a node point.

${}^{n}(L_k)$ = Participating mass ratio for the n^{th} mode and k^{th} degree-of-freedom direction, as determined from Equation App1D-10.

${}^{n}(M)$ = Modal mass for n^{th} mode, as determined by Equation App1D-2.

M_{ik} = Component of mass at i^{th} node point that corresponds to k^{th} degree of freedom direction.

M_k = Total mass in the k^{th} degree of freedom direction, obtained by summing all M_{ik} values.

NK = Total number of directions or degrees of freedom of motion of a node point.

NP = Total number of node points in mathematical model used in dynamic lateral force procedure.

${}^{n}(P_x)$, ${}^{n}(P_y)$, ${}^{n}(P_z)$, ${}^{n}(P_k)$ = Modal participation factor for n^{th} mode and for x-direction, y-direction, z-direction, and k^{th} degree of freedom direction, respectively, as determined from Equation App1D-1.

${}^{n}(S_{ak})$ = Spectral acceleration in the k^{th} direction and corresponding to the natural period ${}^{n}(T)$ of the n^{th} mode.

${}^{n}(T)$ = Natural period of the n^{th} mode.

$(\ddot{u}_{ik})$ = Peak acceleration response in the k^{th} direction at the i^{th} node, and in the n_{th} mode.

$^{n}(\phi_{ik})$ = Component of mode shape for n^{th} mode that corresponds to i^{th} node point and k^{th} degree of freedom direction.

Part 1 Response Spectrum Analysis

The dynamic analysis procedure described in this section uses a response spectrum representation of the seismic input motions. The procedure is applicable to linear elastic building models developed in accordance with the requirements of Section 106.3. It consists of the following steps [U.S. Army, 1986, Clough and Penzien, 1975; Biggs, 1964; SEAOSC, 1977]:

1. Use principles of mechanics to compute the natural period and mode shape for the first N normal modes of the building model, where N is established in accordance with Part 2 of this Appendix.
2. Enter the response spectrum at the natural period of the n^{th} normal mode, $^{n}(T)$, and obtain the corresponding spectral acceleration in the k^{th} direction, $^{n}(S_{ak})$. Here k may be the x or y horizontal direction or the z vertical direction.
3. Use this spectral acceleration together with the mode shapes and the model's lumped mass values to compute the building's modal participation factor $^{n}(P_k)$ for the n^{th} mode and the k^{th} direction corresponding that of $^{n}(S_{ak})$.

$$^{n}(P_k) = \sum_{i=1}^{NP} \frac{^{n}(\phi_{ik})\,(M_{ik})}{^{n}(M)} \qquad \textbf{Eqn. App1D-1}$$

where: $^{n}(\phi_{ik})$ = Mode shape amplitude for the n^{th} mode, i^{th} node point, and k^{th} direction

M_{ik} = Component of mass for the i^{th} node point and the k^{th} direction

$^{n}(M)$ = Modal mass for n^{th} mode

$$= \sum_{i=1}^{NP} \sum_{k=1}^{NK} {}^{n}(\phi_{ik})^2\,(M_{ik}) \qquad \textbf{Eqn. App1D-2}$$

NP = Total number of node points

NK = Total number of directions of motion at a node point

4. Use the above parameters to compute the peak value of any building response quantity when the building is vibrating in its n^{th} normal mode. For example *peak acceleration* at i^{th} node point and in k^{th} direction:

$$^n(\ddot{u}_{ik}) = {}^n(P_k) \bullet {}^n(\phi_{ik}) \bullet {}^n(S_{ak}) \qquad \textbf{Eqn. App1D-3}$$

also, for response in any other direction j:

$$^n(\ddot{u}_{ij}) = {}^n(P_k) \bullet {}^n(\phi_{ij}) \bullet {}^n(S_{ak}) \qquad \textbf{Eqn. App1D-4}$$

lateral force at i^{th} node point and in k^{th} direction:

$$^n(F_{ik}) = (M_{ik}) \bullet {}^n(\phi_{ik}) \bullet {}^n(P_k) \bullet {}^n(S_{ak}) \qquad \textbf{Eqn. App1D-5}$$

base shear force in k^{th} direction:

$$^n(V_k) = {}^n(P_k)^2 \bullet {}^n(M) \bullet {}^n(S_{ak}) \qquad \textbf{Eqn. App1D-6}$$

overturning moment due to lateral forces in k^{th} direction:

$$^n(OM_k) = \sum_{i=1}^{NP} M_{ik} \bullet {}^n(\phi_{ik}) \bullet {}^n(P_k) \bullet {}^n(S_{ak})(h_i \qquad \textbf{Eqn. App1D-7}$$

where h_i is the vertical distance from the i^{th} node point to the base of the building.

5. Repeat Steps 2 through 4 for each of the N normal modes. Then, combine the resulting peak modal response quantities for each mode using an appropriate procedure from Commentary Section 105.5.2, in order to estimate the composite peak response value.

Example Forms of the Dynamic Equations for Common Two- and Three-Dimensional Building Models

Two-Dimensional Building Model

$k = x =$ horizontal displacement

$NK = 1$, $NP = N =$ number of stories

$^n(S_{ak}) = S_{a,n}$ in the $k = x$ direction

$M_{ik} = M_i$ = mass at floor level i

$M_k = M = \sum_{i=1}^{N} M_i$ = total mass

$^n(\phi_{ik}) = \phi_{in}$

$$^{n}(M) = M_n = \sum_{i=1}^{N} \phi_{in}^2 \bullet M_i$$

$$^{n}(P_k) = P_n = \frac{\sum_{i=1}^{N} \phi_{in} \bullet M_i}{\sum_{i=1}^{N} \phi_{in}^2 \bullet M_i}$$

$$^{n}(\ddot{u}_{ik})x_{in} = P_n \phi_{in} S_{a,n}$$

$$^{n}(F_{ik}) = F_{in} = M_i P_n \phi_{in} S_{a,n}$$

$$^{n}(V_k) = V_n = (P_n)^2 M_n S_{a,n}$$

Three-Dimensional Building Model with Rigid Diaphragm Condition

$k=1=x$ $M_{i1}=M_i$

$k=2=y$ $M_{i2}=M_i$

$k=3=\theta$

$M_{i3}=I_i=$mass polar moment of inertia

$NK=3$

$NP=N=$number of stories

x, y, θ are at center of mass M_i.

$$^{n}(\phi_{i1}) = {}^{n}(\phi_{ix})$$

$$^{n}(\phi_{i2}) = {}^{n}(\phi_{iy})$$

$$^{n}(\phi_{i3}) = {}^{n}(\phi_{i\theta})$$

$$^{n}(M) = \sum_{i=1}^{N} \sum_{k=1}^{3} {}^{n}(\phi_{ik})^2 \bullet M_i$$

$$= \sum_{i=1}^{N} {}^{n}(\phi_{ix})^2 \bullet M_i + \sum_{i=1}^{N} {}^{n}(\phi_{iy})^2 \bullet M_i + \sum_{i=1}^{N} {}^{n}(\phi_{i\theta})^2 \bullet I_i$$

$${}^{n}(P_k) = \frac{\sum_{i=1}^{N} {}^{n}(\phi_{ik}) \bullet M_{ik}}{{}^{n}(M)}$$

$${}^{n}(P_x) = \frac{\sum_{i=1}^{N} {}^{n}(\phi_{ix}) \bullet M_i}{{}^{n}(M)}$$

$${}^{n}(P_y) = \frac{\sum_{i=1}^{N} {}^{n}(\phi_{iy}) \bullet M_i}{{}^{n}(M)}$$

$${}^{n}(\ddot{u}_{ij}) = {}^{n}(P_k) \bullet {}^{n}(\phi_{ij}) \bullet {}^{n}(S_{ak})$$

For seismic ground motion in the $k = x$ direction,

$${}^{n}(\ddot{x}_i) = {}^{n}(P_x) \bullet {}^{n}(\phi_{ix}) \bullet {}^{n}(S_{ax})$$

$${}^{n}(\ddot{y}_i) = {}^{n}(P_x) \bullet {}^{n}(\phi_{iy}) \bullet {}^{n}(S_{ax})$$

$${}^{n}(\theta_i) = {}^{n}(P_x) \bullet {}^{n}(\phi_{i\theta}) \bullet {}^{n}(S_{ax})$$

Part 2 Significant Number of Modes

When a structure is vibrating in its n^{th} mode of vibration, the maximum value of the base shear in the k^{th} degree of freedom direction ${}^{n}(V_k)$ can be estimated from the expression:

$${}^{n}(V_k) = {}^{n}(\bar{M}_k) \bullet {}^{n}(S_{ak}) \qquad \textbf{Eqn. App1D-8}$$

where ${}^{n}(S_{ak})$is the spectral acceleration corresponding to the n^{th} mode of vibration and ${}^{n}(\bar{M}_k)$ is an effective mass for computing the modal base shear. It can be shown that:

$${}^{n}(\bar{M}_k) = {}^{n}(P_k)^2 \bullet {}^{n}(M) \qquad \textbf{Eqn. App1D-9}$$

where ${}^{n}(P_k)$ and ${}^{n}(M)$ correspond to the modal participation factor (for the k^{th} direction) and the modal mass respectively for the n^{th} mode, as defined in the main body of Section 106.5.1. From this, ${}^n(L_k)$, the participating mass ratio for the n^{th} mode, can be expressed as:

$$ {}^n(L_k) = \frac{{}^n(\overline{M}_k)}{M_k} \qquad \textbf{Eqn. App1D-10} $$

where M_k is the total mass in the k^{th} direction, and is given by

$$ M_k = \sum_{i=1}^{NP} M_{ik} \qquad \textbf{Eqn. App1D-11} $$

Thus, it is seen that ${}^n(L_k)$ is related to the effective mass used in computing the maximum base shear when the structure is vibrating in its n^{th} mode of vibration.

The above discussion provides a background for interpreting the requirement given in Section 106.5.1 for the minimum number of modes to be included in a response spectrum analysis. The stated minimum of 90 percent of the participating mass represents an approximate basis for providing an acceptable degree of accuracy in the computed value of the dynamic base shear; and hence, the representation of all significant modes. It is based on evaluation of the base shear induced in the k^{th} direction by a constant unit acceleration in that direction $[{}^n(S_{ak}) = 1]$, and makes use of the identities:

$$ M_k = \sum_{n=1}^{N_T} {}^n(\overline{M}_k); \quad \sum_{n=1}^{N_T} {}^n(L_k) = 1 \qquad \textbf{Eqn. App1D-12} $$

where the above summations are over all of the structure's N_T modes of vibration. For this case, the first equation in the preceding paragraph leads to the following expression for the base shear in each normal mode:

$$ {}^n(V_k) = {}^n(\overline{M}_k) \bullet {}^n(S_{ak}) = {}^n(\overline{M}_k) \bullet 1 \qquad \textbf{Eqn. App1D-13} $$

By summing these modal values over all of the structure's N_T modes, the "exact" value of the total base shear becomes:

$$ (V_k)_{NT} = \sum_{n=1}^{N_T} {}^n(\overline{M}_k) \bullet 1 = M_k \bullet 1 \qquad \textbf{Eqn. App1D-14} $$

Now, if N modes are included in the above summation, where $N < N_T$, then:

$$ (V_k)_N = \sum_{n=1}^{N} {}^n(\overline{M}_k) < M_k \qquad \textbf{Eqn. App1D-15} $$

where $(V_k)_N$=estimated base shear value $< (V_k)_{NT}$

Alternatively, if both sides of the above inequality are divided by M_k, the equality can be expressed as:

$$\sum_{n=1}^{N} {}^{n}(L_k) < 1 \qquad \textbf{Eqn. App1D-16}$$

Therefore, the provision given in Section 106.5.1 for including a significant number of modes, N, in the analysis such that:

$$\sum_{n=1}^{N} {}^{n}(L_k) \geq 0.9 \qquad \textbf{Eqn. App1D-17}$$

corresponds to the requirement that the total base shear resulting from these N modes of vibration, $(V_k)_N$, should be within 10 percent of the "exact" value, $(V_k)_{NT}$, i.e.:

$$(V_k)_N \geq 0.9(V_k)_{NT} \qquad \textbf{Eqn. App1D-18}$$

for the special case of a constant unit acceleration in the k^{th} direction $[{}^{n}(S_{ak})=1]$.

Part 3 Combining Modes

The response spectrum analysis procedure described in Section 106.4.1 and in Part 1 of this Appendix provides the maximum responses of the structure when it is vibrating in each of its significant normal modes. However, because these maximum modal responses will not occur at the same time during the earthquake ground motion, it is necessary to use approximate procedures to estimate the maximum composite response of the structure. Such procedures are typically based on an appropriate combination of the maximum individual modal responses, and should account for possible interaction between any closely spaced modal responses that may exist.

A simple and accurate modal combination approach that satisfies this requirement is the Complete Quadratic Combination (CQC) method [Wilson et al., 1981; Der Kiureghian, 1981; Der Kiureghian, 1980; Wilson and Bolton, 1982; US NRC, 1976]. This approach is based on random vibration concepts and assumes that: 1) the duration of the earthquake shaking is long when compared to the fundamental period of the structure; and 2) the design response spectrum exhibits slowly varying amplitudes over a wide range of periods that include the dominant modes of the structure. On this basis, the CQC method leads to the following expression for the structure's maximum composite response, u_k, at its k^{th} degree of freedom:

$$u_k = \left[\sum_{i=1}^{N} \sum_{j=1}^{N} u_{ki} \rho_{ij} u_{kj} \right]^{1/2}$$ **Eqn. App1D-19**

where u_{ki} and u_{kj} correspond to the structure's maximum modal response in its k^{th} degree of freedom when it is vibrating in its i^{th} and j^{th} mode respectively, and ρ_{ij} is the cross-modal coefficient. It is noted that here, u_k, u_{ki}, and u_{kj} are general symbols and may correspond to total acceleration, relative (to base) displacement, interstory drift, base shear, overturning moment, or any other structural response quantity. Furthermore, when computing u_k in accordance with the above expression, the signs of u_{ki} and u_{kj} should be preserved.

The cross-modal coefficient ρ_{ij} as denoted above is dependent on the damping ratios and the natural periods of the i^{th} and j^{th} mode. When the modes have identical damping ratios ξ, ρ_{ij} is expressed as:

$$\rho_{ij} = \frac{8\xi^2(1+r)r^{3/2}}{(1-r^2)^2 + 4\xi^2 r(1+r)^2} = \rho_{ji}$$ **Eqn. App1D-20**

where r is the ratio of the natural period of the j^{th} mode, T_j, to the natural period of the i^{th} mode, T_i (i.e., $r = T_j/T_i$).

From Equation App1D-7, it can be shown that: 1) $\rho_{ij}=1$ when $r=1$; and 2) ρ_{ij} decreases with decreasing r in a manner that is dependent on the modal damping ratio ξ. Furthermore, when the modal periods are well spaced such that:

$$r = \frac{T_j}{T_i} \le \frac{0.1}{0.1+\xi} (T_i > T_j)$$ **Eqn. App1D-21**

then: $\rho_{ij} \approx 0 (i \neq j)$

and the CQC expression for computing the maximum composite response (Equation App1D-19) becomes:

$$u_k = \left[\sum_{i=1}^{N} u_{ki}^2 \right]^{1/2}$$ **Eqn. App1D-22**

that corresponds to the square-root-sum-of-the-squares (SRSS) modal combination approach. This shows that the SRSS approach is a special case of the more general CQC method, and can be applied when the modal periods are sufficiently well spaced in accordance with Equation App1D-21. Furthermore, the quantities ρ_{ij} for $i \neq j$ can be visualized as corrections to the SRSS approach in order to incorporate effects of coupling between closely spaced modes. As shown in Figure App1D-1, these coupling effects become more important as the modal damping ratio increases. Also, these effects are typically important for three-dimensional structural systems, which often have closely spaced frequencies.

In Section 106.2.2, the largest damping ratio that can be considered when developing site-specific spectra is specified to be 0.05. For this damping value, the cross modal coefficient ρ_{ij} can be estimated directly from the curve in Figure App1D-1 for $\xi = 0.05$, or can be obtained by interpolation between successive values given in Table App1D-1. Furthermore, ρ_{ij} can be assumed to be negligible when:

$$r = \frac{T_j}{T_i} \le 0.67 \qquad \textbf{Eqn. App1D-23}$$

References

Biggs, J. M., 1964. *Introduction to Structural Dynamics*, McGraw-Hill Book Company.

Clough, R. W., and Penzien, J., 1975. *Dynamics of Structures*, McGraw-Hill, New York.

Der Kiureghian, A., 1980. "Structural Response to Stationary Excitation," *Journal of the Engineering Mechanics Division*, ASCE, Vol. 106, no. EMO.

Der Kiureghian, A., 1981. "A Response Spectrum Method for Random Vibration Analysis of MDF Systems," *Earthquake Engineering and Structural Dynamics*, Vol. 9, John Wiley & Sons.

SEAOSC, 1977. *Seismic Analysis by Compute,* Electronic Computation Committee of the Structural Engineers Association of Southern California, 1977, SEAOSC, 2550 Beverly Blvd., Los Angeles, California, 90057.

U.S. Army, 1986. "Seismic Design Guidelines for Essential Buildings," Technical Manual, Army TM 5-809-10-1.

Wilson, E. L., et al., 1981. "A Replacement for the SRSS Method in Seismic Analysis," *Earthquake Engineering and Structural Dynamics*, Vol. 9.

Wilson, E.L., and Button, M. R., 1982. "Three-Dimensional Dynamic Analysis for Multi-Component Earthquake Spectra," *Earthquake Engineering and Structural Dynamics*, Vol. 10, John Wiley & Sons.

Table App1D-1. *Cross-Modal Coefficients for Equal Modal Damping Ratios of 0.05 for all Modes*

$r = T_j / T_i$	ρ_{ij}
1.00	1.000
0.95	0.791
0.90	0.473
0.80	0.166
0.70	0.071
≤0.67	≈0.0

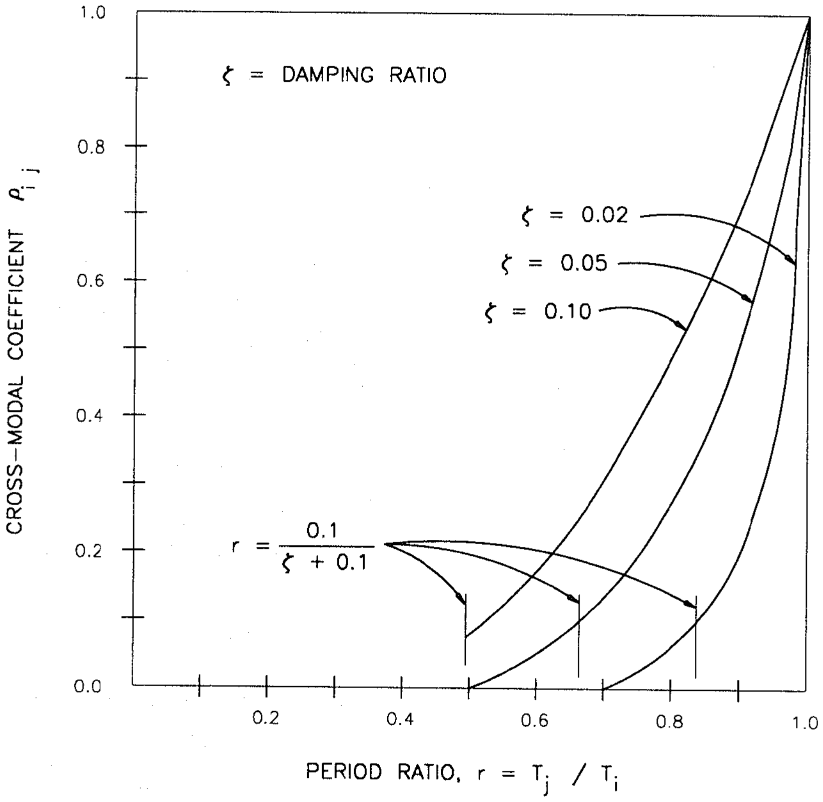


Figure App1D-1. *Effect of damping and period ratio on cross-modal coefficient* ρ_{ij}

APPENDIX 1E

Deformation Compatibility

In order to provide a level of seismic performance consistent with the underlying philosophy of these seismic design provisions, designing and detailing elements not part of the lateral system for deformation compatibility must be considered *equal in importance* to designing and detailing the lateral system for adequate strength and ductility. In a building with a properly designed and detailed lateral system, collapse of the structure can occur if all structural elements are not capable of deforming with the building during the event. Likewise, if certain nonstructural elements in the building are not capable of deforming with the building, the resulting falling hazards may threaten life safety or impede egress from the building. Understanding and complying with these provisions is essential to achieving the seismic performance goals implicit in these Requirements.

Designing for deformation compatibility is a two-part process:

1. Deformation demands must be established.
2. Individual elements and their connections must be assessed for their capacity to deform.

It has been noted by the SEAOC Seismology Committee that both sides of the demand-capacity equation can be assessed using engineering techniques, *but that in certain circumstances* a more effective approach may be to bypass the complex and possibly inaccurate analytical calculations in favor of more conservative design details. In this Appendix, both approaches for demand and capacity assessment are discussed.

Definitions

The provision wording, as set forth in Section 108.2.4: ". . . when subjected to the expected deformations caused by seismic forces" is defined by examination of specific sub-phrases, as follows:

Deformations. Deformations include interstory drift, but also include any other deformation of the structure caused by seismic forces. While interstory drift is the most common deformation value used in practice, other types of deformation need to be considered, such as: 1) vertical racking of structural framing in eccentrically braced frames; 2) shear distortions of concrete coupling beams; or 3) vertical racking of structural bays in dual systems.

Expected Deformations Caused by Seismic Forces. The maximum expected seismic deformations are computed for a ground motion representation having a 10 percent probability of being exceeded in 50 years. For most structures, inelastic response of the structure will occur for this level of ground motion. This inelastic response should be recognized in the computation of the expected deformation. This topic is discussed further below.

". . . when subjected to . . ." If an element is attached to the building in such a way that internal forces are generated in the element when the structure deforms in response to an earthquake, then the element shall be considered "subjected to seismic deformations of the structure."

Scope of the Provision

Section 108.2.4 stipulates that all elements and their connections shall be investigated. The elements included in this scope are structural elements such as columns, beams, walls, slabs, trusses, and bracing that are defined by the engineer as not being part of the lateral system. Also included are nonstructural elements such as stairs, cladding, finishes, utilities, and equipment. The Seismology Committee purposely included all elements of the building because a limited, but definitive list, of elements cannot be delineated in the code.

It is not the intent of the Seismology Committee to require engineering analysis documentation for each and every building component. The intent is to ensure that building designers (engineers and architects) consider all elements and comply *when structural stability and/or life safety are at risk.* Some elements may not be subjected to building deformations in an earthquake; these elements need not be considered. Some elements, while they may be subjected to earthquake deformations and damaged by the imposed deformations and/or induced forces, need not be considered if structural stability and/or life safety are not at risk. A list of common elements and considerations can be found at the end of this Appendix 1E.

Finally, it is recognized that many nonstructural elements are not designed directly by the architect/engineer of record. In these cases, the deformation compatibility requirements must nonetheless be fulfilled and the architect/engineer shall include specific design requirements, including the *maximum expected seismic deformation values*, in the performance design specification.

Deformation Demands

The first step in a deformation compatibility analysis is to determine the deformation demand, denoted herein as D_M. It is the intent of these Requirements to use as a design basis the deformations that can be expected during a major earthquake (probability of exceedance of 10 percent in 50 years), as defined above. This means that the deformations determined by analysis for code level forces ($V=ZICW/R_w$) are not appropriate for purposes of deformation compatibility compliance. Several methods for computing the deformation demands are listed below:

1. Because accurately computing maximum seismic deformations is difficult and very sensitive to method and assumptions, a simple and conservative approach is to assume that interstory drift ratios (story drift to height ratio) are on the order 1.5 to 2 percent (0.015 to 0.020 times the story height). The SEAOC Seismology Committee agrees that an interstory drift of 0.015 times the story height will provide a reasonable level of safety for most structures. With this approach:

   $$D_m = 0.015h$$

   where h = story height.

2. The code provision permits the expected deformations to be computed as the product of the displacements corresponding to code level forces, multiplied by the $3(R_w/8)$ factor. The R_w value would correspond to the value used to design the lateral force resisting system. With this approach:

   $$\Delta_M = 3(R_w/8)\Delta_W$$

   where Δ_W = deformation at $V = ZICW/R_w$

   The code level displacements can be computed using either the equivalent static force procedure or the dynamic analysis procedure. When the dynamic analysis procedure is used, the displacements would be scaled in accordance with Section 106.4.3, then multiplied by the $3(R_w/8)$ factor.

   For purposes of deformation compatibility assessment, the minimum interstory drift must be taken as 0.0025 times the story height. It is noted that research has shown that the $3(R_w/8)$ factor can underestimate the actual inelastic displacement, as discussed under item 3 below.

3. The deformations can be taken directly from an elastic dynamic analysis, provided that neither the input ground motion spectrum (ZIC), nor the results are reduced or scaled to code force level. This approach will generally yield interstory drift ratios on the order of 0.005 to 0.020 for most structures. Generally, this approach, referred to in the literature as "equal-displacements" theory, will yield deformations higher, but more accurate, than those computed using method 2 above. In this approach:

   $$\Delta_M = D_E$$

   where D_E is the unreduced elastic displacement.

   For structures with short fundamental periods and/or with degrading stiffness properties, the above equal-displacements approach may under estimate the actual displacement. In this case, references such as Miranda [1994] and Uang [1991] are recommended.

4. An inelastic time-history analysis, properly executed, will yield the most accurate measure of expected deformations. However, because this approach is not widely used or understood, caution is warranted: accurate nonlinear time-

history analysis is a specialized skill and is not recommended for persons without proper training.

When an engineering analysis is made to determine the maximum expected seismic deformations, proper modeling of the structure is needed. Because deformation incompatibility can have profound life safety implications, it is essential that the deformations *not be underestimated.* The above noted four approaches should be used in conjunction with the following guidelines:

1. **Flexural, shear, and axial deformations of structural elements should be included in the deformation compatibility analysis.** The analysis should include all possible sources of deformation in the structure. Axial and shearing deformations are sometimes excluded in a lateral force analysis, without a loss of accuracy. However, in a deformation compatibility analysis, it is essential that all sources of deformations be included. For example, for frame structures, deformations of the joint region should be included.
2. **P-delta effects shall be included in the deformation compatibility analysis, in accordance with UBC §105.9.** When Section 1628.9 requires that p-delta effects be determined, they shall be included in the analysis of the structure drift.
3. **The deformations of diaphragms shall be included in the deformation compatibility analysis.** Although it is common practice in a force analysis to ignore deformations of the diaphragm between lateral frames lines, significant deformation demands can result from diaphragm deformations. Including these effects may require supplemental analysis in addition to the global system analysis, particularly where computer programs are used that assume rigid diaphragms. Assessing the effect of diaphragm deformations may require additional "hand" analysis to determine the diaphragm displacements in regions remote from the primary lateral system (Figure 1E-1).
4. **The deformations of foundations shall be included in the deformation compatibility analysis.** Although it is common practice to ignore sources of deformation such as rotation of the foundation, significant deformation demands can result from foundation rotations. Rotation of the foundation can increase total deformation values due to rigid body rotations of the lateral force resisting elements of the building. Including these effects may require supplemental analysis in addition to the lateral system analysis, particularly where computer programs are used that assume rigid foundations. Assessing the effect of foundation deformations on deformations of the superstructure may require input from a geotechnical engineer (Figure 1E-2).
5. **The stiffness contributions of structural and nonstructural elements not part of the lateral force resisting system shall be neglected in the deformation compatibility analysis.** When modeling the structure to determine deformation demands, only the primary lateral force resisting system elements may be included in the analysis. Structural and nonstructural elements that are not part of the lateral system may initially contribute to structural stiffness, but because this stiffness may degrade when subject to cyclic loads,

stiffness, but because this stiffness may degrade when subject to cyclic loads, these stiffness properties must be neglected in the demand analysis for deformation compatibility. Any elements assumed to contribute to the lateral stiffness of the structure must be designed for the prescribed lateral forces using all detailing requirements of Chapters 3-9 of these Requirements.

6. **For concrete and masonry elements, flexural and shear stiffness properties shall, as a maximum, be computed as one-half of the gross section elastic stiffness.** While it may be considered conservative to use gross section properties when computing the period of a structure for purposes of determining the minimum base shear, this practice is *not conservative* when analyzing the structure for deformation compatibility demands. Concrete and masonry elements generally crack before code force levels are reached. Further increase in load results in a reduction in effective stiffness. When computing the deformations of the structure, realistic effective stiffness properties must be used. It is generally accepted that one-half of the gross section properties adequately reflects the effective stiffness of a cracked structural member. Other, more accurate, stiffness properties can be used if substantiated by a rational analysis. These reduced stiffness properties must be used for all parts of the lateral force resisting system, including beam and column frame-type elements and for shear wall-type elements.
7. **Torsional effects and orthogonal effects are to be included where required elsewhere in these Requirements.** When torsional loadings are considered or required, and where simultaneous orthogonal loadings are considered or required, the deformation compatibility analysis must include these effects.

Adequacy of Structural Elements

Once the maximum expected seismic deformations have been determined, the engineer must verify the adequacy of all structural elements for the imposed deformation demands. As with estimation of the deformation demand, long and tedious analysis and design can be simplified by conservative detailing: the use of ductile detailing provisions for reinforced concrete, structural steel and strength-based masonry design can usually be accepted as compliant measures. Also, where possible, if structural elements can be isolated from the deformations by the use of special connections, detailed analysis and design of members can be avoided. Finally, many conventionally designed and detailed structural elements and connections have survived strong ground motions without loss of function. Utilizing past records of acceptable performance will aid in reducing computational efforts, but must be used with caution.

Special considerations for assessing structural elements include:

1. **Shear and flexural forces induced in columns may be underestimated by conventional analysis techniques.** In multi-story buildings, higher mode effects can result in a distribution and magnitude of column moments significantly different than those predicted by an equivalent static analysis or by

modal analysis, the structure typically deforms with a "first mode" shape, where all floors displace in the same direction. This first mode deformed shape will induce minimal forces in columns, especially if there is little restraint in the floor framing system.

An actual earthquake may produce an entirely different profile of story displacements during the time-history of ground motion input. For example, due to the higher mode shape response, the upper stories may deflect in an opposite direction with respect to lower story levels. At the floor level where the story drift profile changes directions, columns above and below can be subjected to significantly higher moments than predicted by conventional static force analysis.

Because conventional static analysis techniques only capture the maximum response and do not adequately predict intermediate conditions that may maximize forces in individual column elements, it is recommended that columns be considered as fixed at each end when computing induced flexural forces, unless a study of the individual higher mode response effects or an inelastic time-history analysis is made.

2. **When computing induced forces, the effective clear length or height of the element shall be used, taking into account the stiffening effect of adjoining rigid structural and nonstructural elements.** For example, where cast-in-place parapets or guard rails reduce the clear height of a column, the induced forces will be many times higher than if the column were free to deform along its entire length. This so-called "short column effect" has resulted in many column failures, particularly as reported in the Guam (1993) and Northridge (1994) earthquakes. The best practice is to ensure that rigid nonstructural elements are isolated from the column so that it is free to deform along its entire length.
3. **Concrete columns pose a high risk if the design does not address deformation compatibility.** The forces induced by interstory drift in the building can result in nonductile shear failures and/or compressive strain failures. Either mode of failure, accompanied by cyclic reversals of load, can destroy the column's ability to support purely vertical gravity loads and can result in partial collapse of the structure. Such behavior can usually be avoided if the shear strength is in excess of the shear corresponding to the development of flexural strength of the column, considering the member: 1) is fixed at both ends; and 2) using the maximum possible flexural strength of the column. Secondly, if the column is confined over the potential plastic hinge region with hoops and cross-ties in lieu of the minimum lateral ties, compressive strain failures can be minimized or at least contained. UBC §1921.7 contains specific provisions for concrete members not part of the lateral system and requires the above measures for heavily loaded columns.
4. **Concrete columns loaded in axial compression up to and beyond the "balanced point" (P_{bal}) may have extremely limited deformation**

4. **Concrete columns loaded in axial compression up to and beyond the "balanced point" (P_{bal}) may have extremely limited deformation capacity.** When the factored axial load approaches the P_{bal} condition (approximately $0.4 \bullet A_g \bullet f'_c$), the available ductility of the column may be limited. For such columns, it is not uncommon to find that "ultimate strength" conditions are reached before reinforcing steel has yielded (compressive failure) and this may occur before any noticeable lateral deformation has occurred. It is recommended that extreme caution be exercised with heavily loaded concrete columns.
5. **Yielding in steel and concrete elements is generally acceptable, provided peak strain levels do not exceed critical limit states and provided the element is detailed for ductile yielding.** In structural steel elements, definitive limit state strains are not defined in the literature. Generally, if member curvatures exceed those corresponding to the development of fully plastic moment, members should be fully compact, member or local buckling should be avoided, and tensile strains limited to less than 4 times yield strain. In concrete elements, if compressive strains associated with plastic curvatures exceed 0.003 inches/inch, confinement hoops should be provided in the plastic hinge regions.

Specific requirements for concrete elements are provided in the UBC §1921.7 and in Section 402.13 of these Requirements. Specific provisions for timber, steel, and masonry are not provided at this time and need to be developed. Until such time as specific requirements for these materials are contained in the code, engineers are cautioned to follow the guidelines contained in these Requirements.

Adequacy of Nonstructural Elements

Engineering judgment must be exercised when assessing nonstructural elements. The underlying philosophy of the recommendations, as described in Commentary Section C101 should be used as a guide in assessing nonstructural elements. It is generally accepted that for minor and moderate earthquakes, damage to nonstructural elements should be minimal; this normally requires that nonstructural elements be capable of accommodating code force level deformations without damage. To accomplish this, particular attention to, and specification of, appropriate details is sufficient. Engineering analysis and computations (other than normal drift calculations) are not required.

However, some nonstructural elements of a building that provide a life safety function, or that if damaged pose a life safety threat (including blocking emergency exits from the building), require special analysis, design, and detailing for deformation compatibility. Examples include:

Stairs. A stair stringer is a diagonal strut if rigidly connected at each end to the building. When the structure deforms laterally, the stringer will act as a brace until either the deformation ends or the stringer and/or its connections fail. Stair systems

Cladding. Cladding systems envelope the building in a rigid skin, and if not properly detailed to accommodate deformations of the structure can act as shear panel elements until either the deformation stops or the cladding system and/or its connections fail. Cladding systems must be designed and detailed for deformation compatibility. Failure to accommodate deformations of the structure can result in loss of cladding supports, which in the case of heavy cladding systems such as stone, masonry and concrete finishes, result in falling hazards with serious life safety threats.

Other possibly critical nonstructural elements include: life safety systems (fire suppression systems); viability of places of sanctuary; viability of exit routes; heavy elements that can be dislodged by building deformations but serve no life safety function; seismic joint assemblies and elements crossing seismic joints; glass/glazing/ curtain wall systems; rigid, nonstructural masonry partitions and separations; and others. Individual elements should be reviewed and assessed.

Due to the enormity of nonstructural elements and the various methods to fabricate and erect these elements, finite prescriptive recommendations are difficult to draft. The engineer must, in cooperation with other members of the design team:

1. Review the type and extent of nonstructural elements.
2. Determine if damage or failure of an element or its connections may result in:
   - Falling hazards that threaten life safety.
   - Blocking or loss of emergency exit from the building.
   - Loss of critical life safety systems such as fire suppression systems, fire exhaust systems, sanctuaries, and emergency power.
   - General threat to life safety.
3. Design and detail these elements to accommodate the computed deformations of the structure.

Many nonstructural elements may be damaged beyond repair by building deformations, but prevention of damage is not a code requirement. In these cases, it is advisable for the engineer to notify the client of the approach used, specifically noting that damage may occur.

In some cases, the engineer's client declines to compensate the engineer for design services related to nonstructural elements. In this case, the engineer is obligated to notify the client of:

1. The predicted deformations of the structure.
2. The code requirements for deformation compatibility.
3. The owner's obligation to comply with the governing building code.

Common Elements Considered in Deformation Compatibility Checks

The following list identifies elements that are normally included or excluded from consideration; this list is not complete, nor does it apply to any building under all circumstances.

Element	*Explanation*
Concrete Columns	*Included.* In many building structures, interstory drifts will be large enough to cause post-yield response of concrete columns. If the induced shear force exceeds the member capacity, or if the resulting compressive strains exceed the members capacity, special detailing is required.
Connections	*Included.* In many structural elements, the induced forces may be insignificant to the member but may result in significant damage and loss of strength to the member connections.
Deep Girders	*Included.* Although the connections may be considered "pinned," most deep girder connections possess some degree of fixity. Both the girder and the connections must be investigated. A common approach is to detail the connections for the expected deformation using slotted holes or other means.
Flat Slabs	*Included.* The slab-column joint region is a rigid connection and some frame action will occur. The combined moment/torsion/shear forces induced in the joint by lateral floor drift may exceed the shear capacity of the joint region.
Trusses	*Included.* Similar to girders, seismic deformations can induce large forces in deep trusses if the truss end connections do not isolate the truss from the building deformations. For example, a story deep truss in a moment frame building could be at risk if the diagonals respond as bracing members. Even if the truss is designated as part of the lateral force resisting system and designed according to braced frame provisions, it may not be compatible with the expected deformations; nonlinear response in a primary gravity load member, such as truss chords or diagonals, should be carefully considered, since buckling or rupture of truss diagonals could result in loss of load-carrying capacity.

Element	***Explanation***
Miscellaneous Bracing	*Included.* Various forms of miscellaneous bracing members are used to brace finishes, mezzanines, or other secondary structures. While the bracing may be designed for prescribed code forces, deformations of the structure may induce much larger forces, which could result in loss of member strength, loss of connection, or have secondary consequences. For example, if the brace is connected to a primary structural element and large forces are induced in the brace, the strength and/or stability of the primary structure element could be jeopardized.
Stairs	*Included.* Most stair stringers, if not properly detailed and isolated from seismic deformations of the structure, can act as bracing members. Possible consequences include rupture or buckling of the stringer, failure of its connection, or unexpected forces induced in other portions of the primary structure. Because stairs are an essential life safety element, damage to the stair structure must be avoided.
Cladding	*Included.* Most cladding systems are by their nature stiff and brittle. Because the cladding envelopes the building, it is normally subjected to seismic deformations. While damage to the cladding may occur, designers must ensure that the resulting damage does not result in falling hazards or endanger the primary structure.
Partitions	*Included and Excluded.* If a partition is similar in construction to cladding, special considerations will be needed. Some partitions, such as metal stud and gypsum wall board partitions are light and flexible and do not have a history of seismic hazard if properly designed and built.
Ceilings	*Included and Excluded.* Normally the ceiling system is designed with some isolation from the structure so that deformations of the structure do not induce forces in the ceiling system. If the ceiling system is anchored to the structure in a way that deformations of the structure can cause the ceiling to move, special considerations may be needed.

References

Miranda, 1994. "Strength Reduction Factors for Earthquake Resistant Design," *Earthquake Spectra*. Volume 10, no. 2, Earthquake Engineering Research Institute, Oakland, California.

Uang, C.M., 1991. "Establishing R (or R_w) and C_d factors for Building Seismic Provisions," *J. of Structural Engineering*, ASCE, Vol. 117, no. 1.

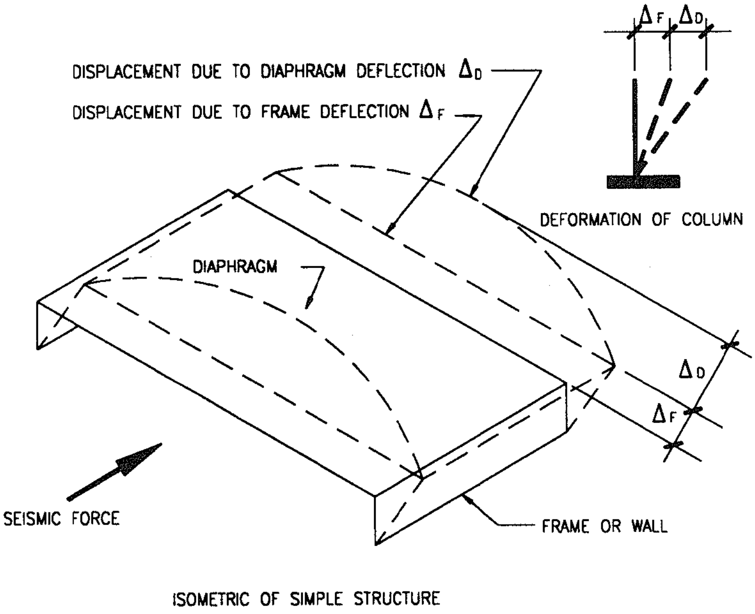

Δ_F
Δ_D
DISPLACEMENT DUE TO DIAPHRAGM DEFLECTION Δ_D
DISPLACEMENT DUE TO FRAME DEFLECTION Δ_F
DEFORMATION OF COLUMN
DIAPHRAGM
Δ_D
Δ_F
SEISMIC FORCE
FRAME OR WALL
ISOMETRIC OF SIMPLE STRUCTURE

Figure App1E-1. *Deformation compatibility consideration of diaphragm deformations*

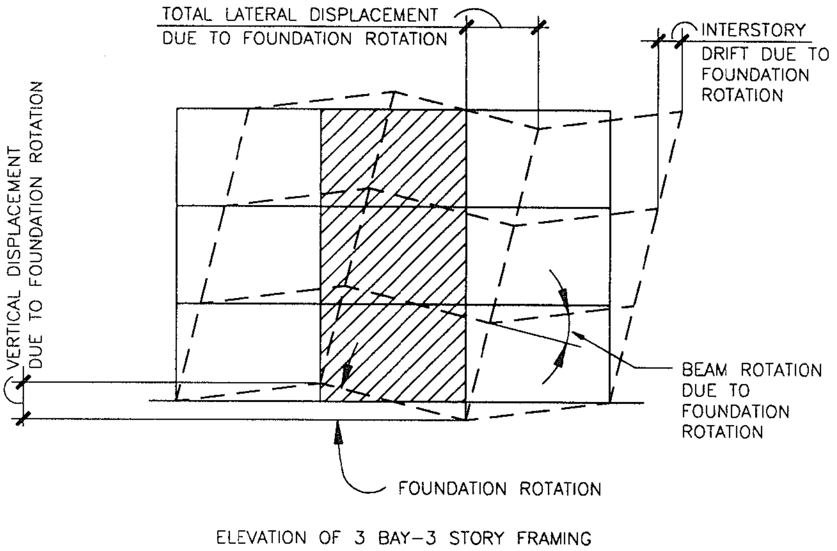


Figure App1E-2. *Deformation compatibility consideration of foundation flexibility*

CHAPTER 2

Structural Tests and Inspections

C201 General Requirements

Past earthquakes have repeatedly demonstrated that good structural performance depends on good design and on having the construction done in conformance therewith. California public schools are an example that good performance can be achieved by buildings that are constructed in conformance with a good design.

C202 Recommended Modifications to the 1994 UBC

C202.1 Special Inspections

C202.1.1 *Modify:* ***UBC §1701.5, Item 11 Piling, Drilled Piers, and Caissons.*** The special inspections listing is provided as a guide to minimum requirements and should be augmented and made comprehensive in the quality control program.

C202.1.2 *Add new item:* ***UBC §1701.5, Item 16 Wood.*** It has been common knowledge that lack of quality control of connections in wood frame construction, both heavy timber and light frame, has been responsible for heavy damage to structures from earthquake and wind in past natural disasters. Major segments of the construction industry, however, have objected to introduction of quality control measures on the installation of these connectors. These measures are introduced specifically in these Requirements because there are no specific provisions in the UBC. Quality assurance measures based on the special inspections called for in this item will mitigate damage in future natural disasters.

C202.2 Structural Observation

Modify: ***UBC §1702 Structural Observation.*** Damage in past earthquakes has too often been traceable to failure to comply with a detail on the construction documents. Better performance can be obtained if the design engineer makes site visits, because he or she is most familiar with the design details and, thus, will more easily detect noncompliance with the construction documents. These site visits do not constitute detailed inspections and do not supplant any aspect of the testing and inspection required by the quality assurance plan and these Requirements.

As has been noted in every past earthquake, including the recent Northridge (1994) event, much of the damage can be attributed directly to the construction not complying with the construction documents and the code. By reviewing the testing and inspection reports as construction progresses, the design engineer has the opportunity to detect early any noncompliance with the quality assurance program and the construction documents, and can take steps to have such noncompliance corrected and prevented in later stages of construction.

C202.3 Design Review

Add new section: ***UBC §1705 Design Review.*** The design review, often referred to as a plan check, is an important step in achieving the goal of good performance. Design review should be welcomed by the design engineer because it affords an opportunity to have the correctness of his or her assumptions verified and to modify possible areas of code noncompliance prior to start of construction. Some jurisdictions have provided for a pre-design review where the design team may pose specific questions in writing and receive written responses to avoid disagreement at the plan check stage.

C202.4 Project Design Peer Review

Add new section: ***UBC §1706 Project Design Peer Review.*** For complex or innovative structural systems it is advisable to have a peer reviewer or peer review panel from the start of the project. A peer reviewer or peer review panel would normally be used at the request of the owner, but some jurisdictions can invoke the review.

Project design peer review is an important step in achieving the life safety goal of the code or, where the owner desires higher performance, the performance goal of the owner. The scope of the review depends, in part, on the type of project under consideration and the performance goals that are set. There are a number of design review documents and recommendations presently in existence. The following are recommended:

1. For general guidance on project design peer review: Chapter 4, "Project Design Peer Review," in *Recommended Guidelines for the Practice of Structural Engineering in California*, second edition. Structural Engineers Association of California, Sacramento, CA, October, 1995.
2. For seismic isolation projects: Section 1660, "Design and Construction Review" in Appendix Chapter 16, *Structural Forces of UBC*, 1994 edition.
3. For repair and strengthening and for design of new welded steel moment resisting frames: Section 4.5, "Qualified Independent Engineering Review" in the *Interim Guidelines: Evaluation, Repair, Modification and Design of Steel Moment Frames*, Report No. SAC-95-02. SAC Joint Venture, Sacramento, CA, August, 1995 (also known as FEMA-267).

C202.5 Quality Assurance

Add new section: ***UBC §1707 Quality Assurance.*** Quality control is the responsibility of the contractor. There is, however, a need for the owner to have assurance that the contractor's quality control program is effective. This is the reason for establishing a written quality assurance program to verify conformance with the construction documents and subject to review by the design engineer(s) of record. The quality assurance program should cover the areas required by UBC §1701.3 and UBC §1701.4, special inspection and standards of quality, and could be more specific and broader.

A requirement to have a written welding procedure prepared by the fabricator-erector, acceptable to the engineer(s) of record and furnished and maintained by the inspector could have led to better performance of building frames in the Northridge earthquake. A quality assurance program affords the design professional an incentive to review his design with respect to areas critical to good performance of the structure. The program can then provide for more detailed quality assurance measures in these areas.

APPENDIX 2A

Quality Assurance

This appendix provides an example of minimum requirements for quality assurance. These requirements are in addition to the testing and inspection required by other sections of these Requirements.

Quality Assurance Plan

A quality assurance plan shall be submitted to the regulatory agency by the engineer of record.

1. **Details of Quality Assurance Plan.** The quality assurance plan (QAP) shall specify the seismic systems designated as subject to quality assurance. The person responsible for the design of a designated seismic system shall be responsible for specifying the QAP applicable to that system. The special inspections and special tests needed to establish that the construction is in conformance with these Requirements shall be included in the portion of the QAP applicable to the designated seismic system.
2. **Constructor Responsibility.** Each constructor responsible for the construction of a designated seismic system, or component, listed in the QAP shall submit a written statement to the regulatory agency prior to the commencement of work on such system or component. The statement shall clearly show the following:

   a. Acknowledgment that he or she is aware of the special requirements contained in the QAP.
   b. Acknowledgment that he or she will exercise control to obtain conformance with the design documents approved by the regulatory agency.
   c. Procedures for exercising control within his or her organization, the method and frequency of reporting, and the distribution of the reports.
   d. The person exercising such control and his or her position in the management of the organization.

Special Inspection

The building owner shall employ an approved special inspector to observe the construction of all designated seismic systems in accordance with the following instructions. Here the term "continuous" means that the inspector is on site in the general area at all times observing the work requiring special inspection.

1. **Foundations.** Continuous special inspection required during driving of piles, construction of drilled piers, and caisson work.
2. **Reinforcing Steel.** Special inspection for reinforcing steel shall be as follows:
   a. Continuous special inspection required during placing of steel in reinforced concrete special moment frames. Verification of correct location of steel in cantilever beams and slabs shall be an included item.
   b. Periodic special inspection required during placing of steel in reinforced concrete and reinforced masonry shear walls and ordinary moment frames.
   c. Continuous special inspection required during welding of reinforcing steel.
3. **Structural Concrete.** Continuous special inspection required during the placing of concrete in drilled piers, caissons, reinforced concrete frames, and shear walls.
4. **Prestressed Concrete.** Continuous special inspection required during placement of prestressing steel, during stressing and grouting operations, and during placement of concrete.
5. **Structural Masonry.** Continuous special inspection required during placement of all masonry units and during all grouting operations for masonry that is part of the seismic resisting system.
6. **Structural Steel.** Continuous special inspection required during all shop and field welding of all multiple-pass welded connections. Periodic special inspection may be specified for single-pass fillet welds according to a procedure outlined in the QAP: This procedure must provide explicit instructions related to the periodic inspection. Periodic special inspection required during high-strength bolting operation for joints.
7. **Structural Wood.** Continuous special inspection required during all field gluing operations. Periodic special inspection required for nailing, bolting, or other fastening.

8. **Architectural Components.** Special inspection for architectural components as follows:
    a. Periodic special inspection required during erection and fastening of exterior panels and interior partitions and ceilings.
    b. Periodic special inspection required during the adhesion or anchoring of veneers.
9. **Mechanical and Electrical Components.** Periodic special inspection required during installation of the anchorage of the following components:
    a. Equipment using combustible energy sources.
    b. Electrical motors, transformers, switchgear unit substations and motor control centers.
    c. Machinery, reciprocating and rotating type.
    d. Piping distribution systems 3 inches or larger.
    e. Tanks, heat exchangers, and pressure vessels.

Special Testing

The special inspector shall be responsible for verifying that the special test requirements are performed by an approved testing agency for the types of work in designated seismic systems listed below.

1. **Reinforcing and Prestressing Steel.** Special testing of reinforcing and prestressing steel shall be as follows:
    a. Sample at fabricator's plant and test reinforcing steel used in reinforced concrete special moment frames and boundary members of reinforced concrete or reinforced masonry shear walls for limitations on weldability, elongation, and actual-to-specified yield and ultimate strength ratios.

       ***Exception*:** Certified mill test certificates may be accepted for ASTM A-706 reinforcing steel, provided that the bars have the raised marking symbol per A-706 or other unique manufacturer identification.

    b. Examine certified mill test reports for each lot of prestressing steel and determine conformance with specification requirements.
2. **Structural Concrete.** Sample at job site and test concrete in accordance with requirement of ACI 318.
3. **Structural Masonry.** Special testing of structural masonry shall be as follows:
    a. Sample and test mortar at the rate of at least once per day, but not less than once for each 2,000 sq. ft. of wall area.
    b. Sample and test grout during each grout pour, but not less than once for each 2,000 sq. ft. of wall area.
    c. When f_m is to be established by prism tests, at least five representative prisms shall be prepared and tested prior to start of work. During construction at least one sample prism shall be prepared per day, but not

less than one sample prism per 5,000 sq. ft. of wall area. Tests shall be performed for compressive strength in accordance with ASTM standards appropriate for the type of unit used.

4. **Structural Steel.** Special testing of structural steel shall be as follows:
   a. Welded connections for special moment frames shall be tested by nondestructive methods conforming to AWS D1.1. All complete penetration groove welds contained in joints and splices shall be tested 100 percent either by ultrasonic testing or by other approved equivalent methods.

      Exception: The nondestructive testing rate for an individual welder may be reduced to 25 percent with the concurrence of the person responsible for structural design, provided the reject rate is demonstrated to 5 percent or less of the welds tested for that welder.

   b. Partial penetration groove welds, when used in column splices, shall be tested by ultrasonic testing or other approved method at a rate established by the person responsible for the structural design. All such welds designed to resist tension resulting from the prescribed seismic design forces shall be tested.

   c. Base metal thicker than 1.5 inches, when subject to through- thickness weld shrinkage strains, shall be ultrasonically tested for discontinuities behind and adjacent to such welds after joint completion. Any material discontinuities shall be accepted or rejected on the basis of criteria acceptable to the regulatory agency with the concurrence of the person responsible for the structural design.

   d. **Reporting and Compliance Procedures.** Each special inspector shall furnish to the regulatory agency, the owner, the persons preparing the QAP, and to the constructor, copies of regular weekly progress reports of observations, noting thereon any uncorrected deficiencies, correction of previously reported deficiencies, and changes to the approved plans authorized by the engineer of record. All deficiencies shall be brought to the immediate attention of the contractor for correction.

Completion

At completion of construction, each special inspector shall submit a final report to the regulatory agency and engineer of record certifying that all inspected work was completed substantially in accordance with approved plans and specifications, as amended. Work not in compliance shall be noted in the report.

At completion of construction, the building constructor shall submit a final report to the regulatory agency and engineer of record certifying that all construction work incorporated into the designated seismic system was constructed substantially in accordance with the design documents and applicable workmanship requirements. Work not in compliance shall be noted in the report.

CHAPTER 3
Foundations

C301 General Requirements

The intent of these Requirements is to provide foundations that have the capability to support the loads that may develop during the inelastic response of the structure induced by major earthquake ground motions. Foundation elements themselves should be designed so that significant inelastic behavior does not occur.

Foundation design loads should include the effects of accidental torsion, as prescribed in UBC §1634.5.

C302 Recommended Modifications to 1994 UBC

C302.1 Allowable Foundation and Lateral Pressures

Modify: ***UBC §1805 Allowable Foundation and Lateral Pressures.*** Seismic shears and overturning effects must be carried into the foundations and a consideration of permissible increased soil capacity for combined effects of vertical and lateral loads should be made as part of the geotechnical investigation. The problem is to provide safety for the short-time loading during an earthquake (or wind) without imposing such restrictions and resulting foundation configuration as to create wide disparity in foundation deflections under normal loading. This disparity could create more damage to the structure than highly increased soil pressure resulting from an earthquake.

The magnitude of horizontal ground pressures and the associated deformations to resist seismic forces should be carefully evaluated; considering soil type, ground motion, and topography.

The advice of a soil and foundation engineer should be sought for the evaluation of:

- Influence of site conditions on seismic lateral forces.
- Possibility of shear failure or densification of the foundation soil during an earthquake.
- Necessity for lateral ties between foundation elements in firm soils.
- Foundation deformations due to overturning forces.

The 1994 UBC includes provisions for consideration of liquefaction potential and its effects that should be accommodated in design. The shear strength of some types of soils, particularly loose, saturated sands, can be substantially reduced by earthquake-induced liquefaction. As a result of liquefaction, foundation soil end bearing capacity and frictional capacity between the soil and the foundation can be reduced. Among several examples of the effects of liquefaction on foundation soil capacity was the severe settlement and dramatic tilting of buildings that occurred during the 1964 Niigata, Japan earthquake. Buildings with both shallow and deep foundations were affected. Other effects of liquefaction that have been observed during earthquakes include lateral spreads and flow slides, floating of lightweight embedded structure, and increased lateral pressures on retaining walls.

The soil and foundation engineer should assess the potential for liquefaction as part of the geotechnical investigation for the proposed structure. In cases where there is a significant liquefaction hazard, this should be considered in selecting an appropriate foundation solution, developing foundation design parameters, and evaluating foundation performance.

Recent publications describing the liquefaction hazard and methods for evaluating it include those by Seed and Idriss [1982], Seed et al. [1983], National Research Council [NRC, 1985], Seed et al. [1985], and Earthquake Engineering Research Institute [EERI, 1986]. The procedure developed by Seed and his coworkers is a widely accepted and economical method for evaluating liquefaction potential. It relates liquefaction potential to the standard penetration resistance (SPT) blow count. A more recent commentary on liquefaction is presented in the 1994 NEHRP provisions [NEHRP, 1994].

C302.2 Requirements for Piles, Pile Caps, and Grade Beams in Seismic Zones 3 and 4

C302.2.1 *Modify:* ***UBC §1809.5.1 General Requirements.*** Bending moments induced in piles by lateral displacements caused by earthquake motions must be considered in pile design.

C302.2.2 *Add new subsection:* ***UBC §1809.5.2.4 Additional Requirements for Piles***

1. No Commentary provided.
2. The reduction factor to transverse reinforcement for confinement of concrete in potential plastic hinge regions is based on the New Zealand concrete design code. Testing by Pam et al. [1990] and Park and Hoat Joen [1990] has demonstrated the adequacy of these Requirements. Uplift testing by Caltrans of a metal shell type pile with vertical flutes and cast-in-place concrete infill resulted in a premature failure by bond between the concrete and steel. Anchorage for uplift loads was made to the concrete only [Chai et al., 1991].
3. Width to thickness ratios of steel plate or pipe elements are prescribed so that the steel pile can develop its plastic moment capacity without local buckling.
4. No Commentary provided.
5. Loss of cover over the confined core will occur at a displacement ductility of approximately 2. The loss of section and reduction of pile capacity is usually

equalized by the overstrength in the core due to confining reinforcement. However, abnormally large cover requirements or low ratios of core depth to gross depth of section are not fully compensated by this rationalization. Therefore, a reduction in the capacity of the pile is warranted for these situations.

C302.2.3 *Add new subsection:* ***UBC §1809.5.3 Pile-Soil-Structure Interaction***

1. Short piles and long slender piles embedded in the earth behave differently when subject to lateral forces and displacements. Long slender pile response depends on its interaction with the soil, represented as an elastic continuum (beam on elastic foundation model), or as a series of unconnected linearly elastic springs (subgrade reaction model). The beam on elastic foundation or subgrade reaction is necessary to obtain realistic pile moments, forces, and deflections [Broms, 1981]. The nonlinear response of the soil should be incorporated into either of the two models through repeated iterations. When the pile length to diameter ratio is 6 or lower, it can be treated as a rigid body. A method assuming a rigid body and linear soil response for lateral bearing is currently in the present codes based on the approach developed by Czerniak [1957]. Analyses of all pile lengths can be made using a variety of computer programs [Reese, 1985] and hand techniques [Duncan et al., 1994]
2. Under lateral loading, substantial degradation of the lateral soil modulus occurs when in-line pile spacing is 3 to 8 pile diameters. This can result in increased pile moments and lateral displacements. This degradation is recognized by several researchers, and state of the art can be found in Ooi et al. [1994].
3. When piles are subject to strong ground motions, bending moments and shears are generated by soil-pile and pile-structure interactions.

   Kinematic or free-field displacement interactions between the soil and pile can occur anywhere along the pile. Such interactions will produce especially large pile curvatures at the interfaces of soft and stiff soil strata. Inertial interaction between the pile and structure results from lateral loads from the structure mass. At the interface of the soft and stiff soil strata, bending moments result primarily from kinematic interaction [ATC-6, 1981; Gazetas et al., 1992]. The maximum moment demand on the pile could occur at either location. Extending the lateral transverse reinforcing for the potential plastic hinge region through this zone will ensure that proper reinforcement will occur in the plastic hinge region. Typically, in analyzing the building response to ground motions, the kinematic interaction is neglected to simplify the analysis.

C302.2.4 *Add new subsection:* ***UBC §1809.6 Special Requirements for Pile Caps and Grade Beams***

1. No Commentary provided.
2. Anchorage of the pile to the pile cap or grade beam for tension or fixity should not fail before developing the pile or soil uplift capacity. It is also desirable that the pile structural capacity exceed the geotechnical failure/pile slip capacity.

This could allow energy dissipating mechanisms, such as rocking, to occur in the soil without structural failure of the pile.

3. Grade beam elements attached to moment resisting columns should not form plastic hinges, unless unavoidable, where detection of failure or distress is difficult, nor should they provide a source of instability for the superstructure. A strong column-weak beam system at the foundation level has little benefit for the superstructure and forces plastic hinges in the grade beams. Column longitudinal reinforcing hooked in footings or pile cap/grade beams are traditionally placed with hooks out. This has been found to substantially reduce the joint shear capacity [Chai, et al., 1991]. The hooks should be turned into the joint in a manner similar to that of a typical beam-column exterior joint.
4. Transverse reinforcing for grade beams and pile caps should extend past the centerline of edge piles sufficiently to develop the reinforcing.

References

ATC-6, 1981. Applied Technology Council, "Seismic Design Guidelines for Highway Bridges."

Broms, B., 1981. "Precast Piling Practice," Thomas Telford Ltd., London.

Caltrans/DFI Seminar, 1993. "Pile Load Test Results at Deep Bay Mud Site Using Various Pile Types," September 9-10, Oakland, CA.

Czerniak, E., 1957. "Resistance to Overturning of Single Short Piles," *Journal of the Structural Division*, ASCE Vol. 83, ST-2.

Duncan, J. M., L.T. Evans, and P.S.K. Ooi, 1994. "Lateral Load Analysis of Single Piles and Drilled Shafts," *Journal of Geotechnical Engineering,* ASCE, New York.

EERI, 1986. Earthquake Engineering Research Institute, *Reducing Earthquake Hazards: Lessons Learned from Earthquakes*, Publication No. 86-02, Oakland, CA.

NEHRP, 1994. *NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings*, Parts 1 and 2, 1994 Edition. Building Seismic Safety Council, Washington D.C., 1994.

NRC, 1985. National Research Council, *Liquefaction of Soils During Earthquakes*.

Ooi, P.S.K. and J.M. Duncan, 1994. "Lateral Load Analysis of Groups of Piles and Drilled Shafts," *Journal of Geotechnical Engineering,* ASCE, New York.

Hoat Joen, P. and R. Park, 1990. "Flexural Strength and Ductility Analysis of Spirally Reinforced Prestressed Concrete Piles," *PCI Journal*, July-August.

Park, R. and P. Hoat Joen, 1990b. “Simulated Seismic Load Tests on Prestressed Concrete Piles and Pile-Cap Connections,” *PCI Journal*, November-December.

Reese, L.C., 1985. *Documentation of Computer Program LPILEI,* Ensoft Inc., Austin, Texas.

Seed, H. B., I. M. Idriss, and I. Arango, 1983. “Evaluation of Liquefaction Potential Using Field Performance Data,” *Journal of Geotechnical Engineering*, ASCE Vol. 109, No. 3.

Seed, H. B., Tokimatsu, K., L.F. Harder, and R.M. Chung, 1985. “Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations,” *Journal of Geotechnical Engineering*, ASCE Vol. 3, December.

Seed, H.B. and I.M. Idriss, 1982. *Ground Motions and Soil Liquefaction During Earthquakes*, Earthquake Engineering Research Institute, Oakland, CA.

CHAPTER 4

Reinforced Concrete

C401 General Requirements

It is the intent of these Requirements to provide tough, ductile behavior in reinforced concrete by preventing brittle modes of failure that might lead to collapse of portions or all of the structure. Tests and studies indicate that this can generally be attained by ensuring the yielding of tensile reinforcement prior to compression, shear, or bond failures. Further, it is intended that ductile behavior occur at levels of deformations that may realistically be anticipated in Seismic Zones 3 and 4, and that this ductile behavior would include the ability to withstand a number of significant reversals of loading. A major objective of these Recommendations is to further ensure both strength and ductility by the specification of constructible reinforcing details that will allow proper placement and consolidation of concrete in the members and connections.

The Requirements of Chapter 4 represent minimum requirements governing the design of all reinforced concrete structural members in Seismic Zones 3 and 4. Two noteworthy exceptions, however, are prestressed and precast concrete systems. The lack of laboratory testing and postearthquake damage assessments of precast and prestressed systems limits current knowledge, and has prevented the development of related seismic design requirements. However, such systems may be used to resist seismic forces and/or deformations in Zones 3 and 4 provided satisfactory evidence in the form of experiments, testing, and analysis based on established engineering principles is given to show that they offer the same toughness, ductility, strength, stiffness, stability, durability, energy absorption, and energy dissipation capabilities as monolithic cast-in-place concrete. Connections must be designed based on established engineering principles and must, in some cases, provide adequate strength and ductility for satisfactory performance under cyclic loads and must be capable of developing the probable strength of yielding portions of the structure.

C402 Recommended Modifications to 1994 UBC

C402.1 Details of Reinforcement

C402.1.1 *Modify*: ***UBC §1907.1.3, Item 4 Stirrups and Ties.*** An editorial change.

C402.1.2 *Modify:* ***UBC §1907.10.5.6 Hooks and Column Ties.*** An editorial change to clarify the requirements.

C402.2 Strength Reduction Factors

C402.2.1 *Modify*: ***UBC §1909.3.4.1.*** An editorial change to clarify that when the shear force at nominal flexural strength is being computed, it is essential that the highest nominal moment strength be used in the calculation. The factored dead, live, and seismic contributions to axial load must be considered.

The "most critical factored axial load" is not necessarily the highest possible axial load. The most critical factored axial load is the axial load that results in the maximum nominal flexural strength in the member. If the factored axial loads are greater in magnitude than the axial load corresponding to the balanced condition, then the axial load corresponding to the balanced condition should be used.

C402.2.2 *Modify:* ***UBC §1909.3.4.2.*** The entire provision permitting a $\phi = 0.5$ for columns in lieu of confinement is deleted and replaced by an unrelated provision for chords and collectors of diaphragms. It is now recognized that increasing the strength of a column (in lieu of providing the special detailing requirements of UBC §1921.4.4) is not a reasonable substitute for adequate transverse reinforcement in regions of high seismicity. The former $\phi = 0.5$ strength reduction factor is no longer permitted.

C402.3 Limits of Section

Modify: ***UBC §1910.8.4 Limits of Section.*** This revised requirement recognizes that in Seismic Zones 3 and 4, inelastic response of structural members is likely, and therefore the design and detailing of the structure must be based on the actual section of the member. This revision now prohibits the practice of using a reduced effective section for the design of members in Seismic Zones 3 and 4. Where the actual strength of a member must be computed, as in portions of UBC §1921, the strength determination must account for the actual cross-section. Where the minimum reinforcing content is specified in portions of UBC §1921, the actual section dimensions should be used. Where special transverse reinforcing is required by §1921, the amount of volumetric transverse reinforcing must be based on the actual cross-section.

C402.4 Circular Columns

Add new subsection: ***UBC §1911.5.6.9 Circular Columns.*** Specific requirements for shear strength assessment of circular members are provided in the main body of UBC §1911. The area assumed to contribute to V_c is provided, and a prescriptive equation for the contribution of steel to shear strength is provided.

C402.5 Reinforced Concrete Structures Resisting Forces Induced by Earthquake Motions

Add notations: ***UBC §1921.0 Notations.*** New notations have been added for use in Section 402.9.

C402.6 Flexural Members of Frames

Modify: ***UBC §1921.3.1 Scope.*** This item requires that any frame member resisting axial gravity loads conform to UBC §1921.4. It is retained from the 1990 SEAOC Recommendations, but has not been submitted by the Seismology Committee to ICBO as a Code Change Proposal. Refer to Section C404.1 for additional discussion.

C402.7 Frame Members Subjected to Bending and Axial Loads

C402.7.1 *Modify:* ***UBC §1921.4.1 Scope.*** This item is related to item 402.6 and requires that any frame member resisting axial gravity loads conform to UBC §1921.4. It is retained from the 1990 SEAOC Recommendations, but has not been submitted by the Seismology Committee to ICBO as a Code Change Proposal. Refer to Section C405.1 for additional discussion.

C402.7.2 *Modify:* ***UBC §1921.4.3.2 Splice Locations.*** Refer to Section C405.3 for discussion of this item.

C402.8 Shear Strength

Modify: ***UBC §1921.5.3.1.*** The definition of *Aj* is changed to be consistent with the ACI-352 data used to establish joint shear strength values. This definition is also consistent with ACI 318-95. This item has not been submitted to ICBO as a Code Change Proposal. Refer to Section C406.2 for additional discussion.

C402.9 Design of Shear Walls for Flexural and Axial Loads

Modify: ***UBC §1921.6.5.*** Shear wall design provisions have been significantly revised to improve overall wall performance and to reduce reinforcement congestion at boundary elements. Refer to Section C407.2 for discussion of this item.

C402.10 Boundaries of Structural Diaphragms

Add new subsection: ***UBC §1921.6.6.3.*** The minimum cover surrounding chord and collector reinforcement in concrete diaphragms is increased. This item, adopted by ICBO for the 1996 UBC supplement, was developed in response to the 1994 Northridge earthquake damage where spalling of concrete around these types of bars rendered the bars ineffective. Refer to Section C407.3 for discussion of this item.

C402.11 Coupling Beams

Replace: ***UBC §1921.6.9.*** Design strength and detailing requirements for coupling beams are updated. Foremost of the changes is the requirement for confinement of diagonal reinforcement under certain conditions. However, the change also permits "coupling beams" to be neglected when the remaining system has sufficient strength—in effect relaxing the detail requirements under certain conditions. This item, adopted by ICBO for the 1996 UBC Supplement, replaces the 1994 UBC provisions. The new provisions will result in details consistent with previous test specimens with recommendations proposed by Paulay [1975, 1992] and with 1994 NEHRP recommendations. Refer to Section C407.5 for additional discussion.

C402.12 Minimum Thickness of Diaphragms

Modify: ***UBC §1921.6.9.11.*** The minimum thickness of concrete toppings on precast floor systems is increased. This item, adopted by ICBO for the 1996 UBC Supplement, was developed in response to 1994 Northridge earthquake damage where the poor performance of lightly topped precast floor systems led to the loss of bond around reinforcing steel of the diaphragm. This item is related to item 402.10. Refer to Section 407.7 for additional discussion.

C402.13 Frame Members Not Part of the Lateral Force Resisting System

Replace: ***UBC §1921.7.*** Deformation compatibility requirements for "nonseismic" concrete frame elements are updated with a significant increase in detailing requirements, primarily increased confinement of members supporting axial gravity loads. This item, adopted by ICBO for the 1996 UBC Supplement, was developed in response to 1994 Northridge earthquake damage where the poor performance of "nonsiesmic" frame members led to partial or total collapse of structures. Similar changes were also made to ACI 318-95. Refer to Section C408 for additional discussion.

Additional Commentary

The following Commentary discusses selected sections of the 1994 UBC. Sections that do not reference a UBC section or paragraph present discussion of specific topics and do not refer to any specific UBC requirement.

C403 General Requirements

C403.1 UBC §1921.2.3 Strength Reduction Factors

There is a lack of evidence to indicate that the degree of conservatism specified in ACI 9.3.4 and 21.2.3 is necessary for diaphragms, since the consequences of overstress are generally less severe for diaphragms than for walls. SEAOC recommends deletion of the 0.6 strength reduction factor for typical cast-in-place diaphragms. However, because of the general lack of experience with topping slabs used as diaphragms over precast concrete members, the strength reduction factor of 0.6 is recommended to compensate for uncertainties in predicting their behavior. For the case of offset shear walls (Irregularity Type D in Table 104.6 in Requirements), where the diaphragm is required to transfer wall shear from the vertical plane of one wall to the plane of the offset wall, the diaphragm should be considered as a part of the shear wall and the $\phi = 0.6$ value should be used. UBC §1909.3.4.1, which differs from ACI §9.3.4.1, reflects the SEAOC recommendation.

C403.2 UBC §1921.2.4.2 Lightweight Concrete

There is evidence suggesting that lightweight concrete ranging in strength up to 12,490 psi can attain adequate ultimate strain capacities [Kaar et al., 1978]. Testing to examine the behavior of high strength, lightweight concrete under high-intensity cyclic shear loads, including a critical study of bond characteristics, has not been extensive in the past. However, there are test data showing that properly designed lightweight concrete columns, with concrete strength ranging up to 6,210 psi, maintained ductility and strength when subjected to large inelastic deformations from load reversals [Rabbat et al., 1986]. It was felt that a limit of 6,000 psi on the strength of lightweight concrete was advisable, pending further testing of high strength lightweight concrete members under reversed cyclic loading.

C403.3 UBC §1921.2.5 Reinforcement in Members Resisting Earthquake Forces

SEAOC does not recommend the use of high strength reinforcing steel with yield strength in excess of 60 ksi for the reasons stated below.

First, tests have shown that bond characteristics greatly influence the flexural stiffness and ductile behavior of reinforced concrete members and their joints in the inelastic range, with bond stresses becoming much lower and bond-slip increasing significantly in the postyield range over those in the elastic range. Second, cyclic tests of reinforced concrete joints indicate that the bond-slip increases further in the inelastic range with an increase in the yield strength of the reinforcing steel, which in turn further reduces the available bond stresses.

In order to compensate for this reduction in bond stresses and enhance ductility when using high strength reinforcing steel, high strength concrete has been experimented with to meet the higher bond demand. Unfortunately, there is a lack of research conducted in the U.S. on the inelastic, cyclic response of high strength reinforcing steel, and in fact little research is available for high strength reinforcing subjected to monotonic loading.

However, since 1988, a major effort has been undertaken by Japan's Ministry of Construction to direct a comprehensive program in development, design and construction procedures for high strength and ultra-high strength reinforcing steel and concrete to allow highrise, or super highrise structures to be built more economically.

This five-year program has produced an enormous amount of experimental and analytical data related to high strength concrete with specified compressive strength in the range of approximately 4.5 to 18 ksi and high strength reinforcing steel with yield strength of approximately 60 to 180 ksi.

For example, results from testing of standard hooks with yield strength of 120 ksi show proportionately adequate pullout capacity if they are embedded in 12 ksi, or stronger, concrete. Cyclic loading of high strength reinforced concrete columns with low axial stresses exhibit excellent ductile structural response when confined with high strength lateral ties, provided the full capacity of transverse reinforcement is normalized by the strength of concrete [$(f_y / f'_c \approx 0.18$ (see UBC §1921 for definition of terms) has been suggested, as a lower bound].

Despite many encouraging results, several areas may need further investigation and refinement, including:

- Splicing methods using gas-pressure welding or enclosed inert gas welding (popular in Japan) seem to be inadequate to develop ultimate strength of high strength reinforcing steel.
- Smaller residual displacement and energy dissipation were observed when high strength longitudinal bars were tested (as opposed to normal strength longitudinal bars) in columns subject to shear and bending, with all other elements remaining the same, such as confinement reinforcement, strength of concrete, and product of longitudinal bar cross sectional area and its yield strength.
- Concrete splitting failure occurred at several locations along the height of high strength columns with small shear span-depth ratios when subject to high axial stresses, perhaps due partly to the low tensile strength of concrete.
- Structural walls containing high strength materials have also been subjected to extensive testing under simulated gravity and cyclic lateral loads. The observed modes of failure indicate poor energy dissipation and ductility as indicated by pinching of the hysteretic loop. [Sugano et al., 1988; Okamura et al., 1987; Kaku et al., 1991; Kabeyasawa et al., 1992; Miura et al., 1990; Aoyama et al., 1992].

It is perhaps noteworthy that physical and chemical properties of some materials used in Japan may vary from those of their counterparts in the U.S. For example, the type of superplasticizers or other admixtures used in high strength concrete, the difference in rebar elongation characteristics, and other variables in testing criteria and data interpretation often necessitate verification of such results using U.S. materials and standards. Nonetheless, further experimentation, together with new theoretical and design developments in seismic applications of high strength reinforcing steel and concrete may pave the way for structures using high strength materials; however, in the meantime, the limitations in these Requirements imposed on strengths of materials are consistent with the current state of the art in the U.S.

C403.4 UBC §1921.2.7 Load Factors

At present, there is no consensus among SEAOC, ACI, and ASCE regarding the appropriate load factors to be used in combining dead, live, and earthquake loads for the strength design of reinforced concrete members. The lack of statistical data with which to critically examine load combinations, including earthquake effects, has led SEAOC to retain the 1980 Blue Book load factors of $1.4D$ and $1.4L$ in lieu of the ACI values of $1.05D$ and $1.28L$. The higher load factors for dead and live loads provide increased protection for members carrying vertical loads. In addition, these load factors compensate, in part, for the effects of vertical seismic accelerations not otherwise explicitly considered in design.

C404 Flexural Members of Frames

C404.1 UBC §1921.3.1 Scope

The requirements of UBC §1921.3 are intended to apply to horizontal elements of frames such as beams or girders, resisting lateral forces induced by earthquake motions. UBC and ACI permit these requirements to be applied to columns supporting axial loads below $A_g f'_c/10$. SEAOC believes that all members of the lateral force resisting system that carry gravity load by axial compression (i.e., columns) should meet the proportioning and reinforcement requirements of UBC §1921.4 (see Commentary Section C405).

The geometric constraints for flexural members are based primarily on past practice. The limit on the width-to-depth ratio is intended to provide compact cross-sections that have a reasonably low risk of lateral instability in the nonlinear range of response. The maximum width limitation is related to the problem of efficient transfer of moment from beam to column. This limitation intentionally prohibits the use of a flat plate or flat slab strip for a beam element in a seismic resisting frame, unless special details are incorporated in the slab structure. It should be noted that although it may be possible to provide the necessary flexural strength in that portion of the slab permitted to be designed as a beam, it is likely that drift will be excessive.

C404.2 UBC §1921.3.2 Longitudinal Reinforcement

The limitations on reinforcement are intended to achieve ductile behavior in beams and girders. The minimum reinforcement ratio, $200/f_y$, is consistent with the desire to provide minimum reinforcement strength greater than the tensile strength of the concrete in an uncracked section. The maximum reinforcement ratio, 0.025, is intended to ensure that the flexural member has adequate rotational capacity resulting from yielding of the reinforcement prior to compression crushing of concrete.

A practical limitation on the amount of reinforcing permitted in a flexural member results from the congestion of bars. The amount of column main steel and special transverse reinforcing in the joints, the space required for steel passing through the joint from orthogonal beams, and the requirements for development of beam reinforcement make necessary a careful study of the bar layouts, including splices. Field tolerances in steel placement must be considered and adequate access must be provided for proper concrete placement and consolidation.

In spite of the fact that moments at both ends of a beam or girder may be negative under specified seismic load combinations, continued lateral displacement of the structure due to maximum expected levels of ground motion will add positive moment to either end of a beam. To ensure strength and ductility under these extreme displacement conditions, positive moment strength equal to at least 50 percent of the negative moment strength must be provided at both ends. This additional reinforcement also increases the flexural ductility under negative moments when this reinforcement acts in compression [Park and Paulay, 1975].

C404.3 UBC §1921.3.3 Transverse Reinforcement

Transverse reinforcement is required in flexural members for two reasons: 1) to provide the member with adequate shear strength; and 2) to confine the concrete and restrain the longitudinal reinforcement against buckling in zones where there is a possibility of yielding under earthquake effects. Reinforcement for the first requirement is specified in UBC §1921.3.4 and, for the second in UBC §1921.3.3. The amounts of transverse reinforcement required for shear and confinement are not required to be additive.

C404.4 UBC §1921.3.3 Seismic Hooks

A stirrup, as defined by ACI, may have 90-degree hooks, whereas a "seismic hook" has a 135-degree bend anchored in the confined core of a member, since 90 degree hooks can lose their effectiveness after spalling of the concrete cover. SEAOC recommends seismic hooks on stirrups to restrain longitudinal reinforcement from buckling and to provide a nominal amount of confinement throughout beams and girders that are part of the lateral force resisting system.

C404.5 UBC §1921.3.4.1 Shear Strength Design Forces

The notation M_{pr} is introduced to represent the probable flexural strength of beams and columns. M_{pr} includes a strength reduction factor of $\phi = 1.0$ and an allowance for overstrength of reinforcing steel ($1.25\ f_y$).

This UBC provision differs from the companion ACI provision (§21.7.1.1) in that the word "factored" is deleted before the phrase "tributary gravity loads." In other words, ACI stipulates that the total shear is the shear at flexural yield plus the shear due to "factored" gravity loads. Recognizing that UBC gravity load factors for the seismic load combination are higher than the corresponding ACI load factors, SEAOC believes that the resulting beam design shear force would be unnecessarily conservative. The use of unfactored tributary gravity loads in combination with the probable beam flexural strengths represents a rational approach for the calculation of design shear force.

C405 Frame Members Subjected to Bending and Axial Loads

C405.1 Scope

SEAOC has expanded the scope in UBC §1921.4.1 to include all framing members of the lateral force resisting system supporting gravity loads by axial compression, rather than limiting it to members supporting an axial compressive force exceeding $A_g f_c'/10$. As a result, all columns will be subject to the same proportioning and reinforcement requirements.

C405.1.1 ***UBC §1921.4.1.1 Dimensions of Sections.*** Geometric constraints for columns follow previous practice and approximate the proportions used in tests. In the preliminary sizing of columns, the engineer is alerted to consider the additional geometric constraints imposed by considerations of shear in the joint (UBC §1921.5) and development of beam reinforcement within the joint.

C405.2 UBC §1921.4.2 Minimum Flexural Strength of Columns

The intent of UBC §1921.4.2 is to reduce the likelihood of flexural yielding in columns and to restrict inelastic action to beams that are typically subjected to small axial forces. This provision also increases the likelihood that a structure will not form a column sway collapse mechanism.

C405.3 UBC §1921.4.3.2 Splice Locations

There is some substantiating evidence indicating that it may be possible to construct a frame joint that performs satisfactorily with the longitudinal bar lap splice near the joint. It is felt, however, that due to common construction practices, this should not be recommended detailing. The lapped longitudinal reinforcing bars and the required confinement reinforcement congest the region of the splice and make concrete placement more difficult. It is therefore recommended that lap splices be placed away from potential regions of hinging to reduce any adverse effects of reduced concrete quality.

SEAOC adds additional requirements to UBC §1921.4.3.2 (see Requirements Section 402.7). The length of a Class A tension lap splice designed in accordance with UBC §1912.15 is subject to the application of a number of modification factors in UBC §1912.2. One such factor permits the designer to reduce the splice length by providing reinforcement in excess of that required by analysis. Since actual earthquake forces may require full development of each bar, this reduction should not be taken in proportioning the splice.

C405.4 Transverse Reinforcement

C405.4.1 ***UBC §1921.4.4.1, Item 5 Excessive Cover, Minimum Reinforcement.*** Minimum detailing requirements are prescribed for columns, which for architectural or other reasons, are designed with 4 or more inches of concrete beyond the confined core. Reinforcement is required to prevent the spalling of large volumes of concrete and the subsequent sudden loss of column stiffness and/or falling hazards.

Item 6. Point of Contraflexure. When the dynamic force procedure is used, the location of the point of contraflexure may be determined from the results of the equivalent static force procedure. The results of the dynamic analysis will not provide the necessary information if the particular method of modal combination (such as SRSS) does not retain the algebraic signs of the column end moments.

C405.4.2 ***UBC §1921.4.4.4 Transverse Reinforcement, Extent.*** If the moment gradient over the height of a column is relatively flat, the hinging region may extend over considerably more than l_o as defined in ACI §21.4.4.4.

C405.4.3 UBC §1921.4.4.5 Transverse Reinforcement at Discontinuities. The requirement for special transverse reinforcement at discontinuities is intended to ensure that the forces developed in a discontinuous wall can be transferred to its supporting element. The reduced anchorage specified where bars extend into a footing or mat is in acknowledgment of the beneficial confinement offered by the mass concrete.

C405.4.4 UBC §1921.4.4.7 Transverse Reinforcement at Columns. The maximum load that can ever be imposed on a column is that resulting from application of full dead and live loads concurrent with development of flexural strengths at both ends of every beam or girder framing into the column at all levels above that in question. While it is recognized that this may be an improbable condition, it is nevertheless obvious that if the strength of the column exceeds this value, no overload is possible and therefore confinement over the full length is not necessary. Considering the low probability that this load condition would occur, and recognizing the inconvenience of making several biaxial bending computations for every column in the building, it is assumed to be sufficient to consider maximum applied beam moments on the individual column section in question as occurring about only one axis at a time. For simplicity in calculations, the strength of the column may be taken as the axial strength at minimum eccentricity, i.e., $0.8P_o$.

C405.5 UBC §1921.4.5.1 Shear Strength Design Forces

The notation M_{pr} used for flexural members also applies to columns. The upper limit on shear force occurs when plastic hinges form in the column at the top and the bottom of a column. The UBC permits a lower design shear force, computed as the column shear force present when beam plastic hinges form at the top and the bottom of the column. This permitted reduction should be used with caution. Elastic methods of analysis may not conservatively predict maximum column shear forces. Plastic and nonlinear methods of analysis provide a more reliable measure of maximum column shear forces. Caution is also warranted as the code does not thoroughly address shear strength of concrete columns subjected to flexural yield and/or net tension. In summary, the column design shear force should be based on the maximum forces that can be delivered to the column, corresponding to the governing yield mechanism.

C406 Joints of Frames

C406.1 UBC §1921.5.1 General Requirements

In order to ensure that the required energy dissipation capability can be developed in reinforced concrete frame structures, careful consideration must be given to the design and detailing of beam-column joints. Such joints should be designed and detailed to:

1. Preserve the integrity of the joints sufficiently so that the strength and deformation capacities of the connecting beams and columns are developed.

2. Prevent excessive degradation of joint stiffness under seismic loading by minimizing cracking of the joint concrete and by preventing the loss of bond between the concrete and longitudinal beam and column reinforcement passing through the joints.
3. Prevent brittle shear failure of the joint.

The first of these requirements can be met by adequate confinement of the joint concrete. The second can be dealt with by providing adequate transverse joint reinforcement (ties or hoops) and by adequate anchorage or development of the longitudinal beam and column reinforcement. The third requirement dictates that the joint be provided with adequate shear strength to resist the forces imposed on it by the connecting members.

At present, there is some disagreement among researchers regarding the determination of joint shear strength. The reasons for the differences include disagreement on the mechanism of joint shear resistance and the deformation requirements for concrete frame structures. Current thinking is that the shear strength of joint cores is not very sensitive to joint shear reinforcement. In the ACI design procedure, the design horizontal shear force on the joint core is limited to an allowable nominal value and the amount of transverse reinforcement is determined by confinement considerations. The engineer is reminded, however, that these methods and recommendations are based on past experience and laboratory data, and are subject to revision as better data become available.

C406.1.1 ***UBC §1921.5.1.4 Minimum Dimensions of Column.*** The loss of bond between longitudinal beam reinforcement and joint concrete under cyclic load reversals causes a significant increase in frame deformations, or drift, which can lead to frame instability from p-delta effects. In order to prevent excessive degradation of joint stiffness under seismic loading, it is necessary to minimize this loss of bond. The minimum column size requirement is intended to reduce the loss of bond between beam longitudinal reinforcement and the joint concrete at deformation reversals beyond the yield point of the steel. Additional measures can be taken to improve the condition for anchorage and bond development at beam-column joints. These include forcing the location of beam plastic hinges away from the column face. This can be accomplished by designing hunched beams, cutting off or hooking some of the longitudinal beam reinforcement some distance away from the column face, or by bending and diagonally crossing some of the longitudinal beam reinforcement at a distance from the column face.

Nearly all of the research conducted on beam-column joints has been performed on specimens constructed of normal weight concrete. Because of the reduced bond strength of lightweight concrete, the minimum column size required should be increased where columns are constructed with lightweight aggregate. Although no specific design recommendations can be given, the modification factors for lightweight concrete presented in ACI 12.2 may be appropriate to apply to the $20d_b$ minimum column dimension limitation until further information is available.

C406.2 UBC §1921.5.3.1 Shear Strength in Joint Region—Definition of A_j

The definition of A_j makes it consistent with the areas used in test reports from which the strength formulas were developed.

C406.3 UBC §1921.5.4.1 Development Length for Reinforcement in Tension

The development length for a bar with a standard 90-degree hook is provided by UBC Eqn. 21-5. Eqn. 21-5 is derived from the provisions of UBC §1912.5 by multiplying the hook development length specified therein by factors of: 0.7 for cover exceeding 2.5 inches; 0.8 for a connection confined by ties spaced at not more than three bar diameters; 1.25 for the ratio of actual bar yield stress to specified yield stress; and 1.1 to account for cyclic load effects.

In recognition of the rather limited amount and variable behavior of one-way load tests on hooks [Zsutty, 1985], the combined factors of 0.7 and 0.8 for cover and confinement may need further consideration. Reversed cycles of tension and compression yield may also be more destructive to bond than would be represented by the cyclic load factor of 1.1. Also, since reinforcing steel can have a total strain-hardening ratio in excess of the 1.25 factor, a value nearer to the ratio of ultimate strength to specified yield strength may be more appropriate. Recent cyclic load tests [Hawkins et al., 1987] have shown that pull-out failures can occur at hook development lengths only slightly less than those given by UBC Eqn. 21-5. The engineer should consider the consequences of hook pull-out in exterior beam-column joints and provide extra development length, especially where seismic load is the major part of the total load combination. The $20d_b$ minimum column dimension in UBC §1921.5.1.4 can serve as a guide.

C407 Shear Walls, Diaphragms, and Trusses

UBC §1921.6 General. Seismic design of shear walls is similar to seismic design of frames in that details, sizes, and proportions should be selected to ensure that ductile yield mechanisms occur first, thereby limiting forces tending to cause brittle failures. Unlike the design of frame members, however, the proportions of walls often make it impractical to design for ductile flexural yield mechanisms. Therefore, in order to achieve adequate energy dissipation, detailing provisions are tied to the maximum shear stress. Two curtains of anchored shear reinforcement are required for walls with factored shear stresses in excess of $2\sqrt{f_c'}$, and confined boundary members must be provided whenever significant compressive strains are anticipated at the edges of a wall. With such boundary zone detailing, wall panels are expected to form progressive patterns of closely spaced diagonal cracks for each load direction. This diagonal pattern of cracks is well contained by the panel reinforcement and is stabilized out-of-plane by the presence of well-confined boundary members. It has also been shown that boundary members greatly increase the rotation capacity of walls and add significantly to their shear strength by acting as stiff, strong dowels.

Web crushing generally limits the shear strength of walls with boundary members. Under cyclic input, walls designed in accordance with UBC §1921.6 can form stable nonlinear softening hysteresis loops with progressively increasing damping. Such behavior provides an effective means of controlling seismic response and of dissipating energy. In order to achieve this intended performance, however, joints between walls and diaphragms must be properly detailed to ensure adequate force transfer. Furthermore, construction joints in walls must be carefully inspected for adequate roughening and cleaning to ensure monolithic behavior under cyclic loading (see Paulay [1986] and Oesterle et al. [1980] for additional information concerning shear walls).

Chord and Collector or Drag Reinforcement. One critical detailing requirement too often overlooked in design involves the development of chord and collector or drag reinforcement. Horizontal chord and collector or drag reinforcement from floor and roof diaphragms must be carried far enough into a wall for that section of the wall to resist the drag. Furthermore, the reinforcement must be continued beyond that section for a distance sufficient to develop the strength of the reinforcement without damage to the concrete. Preferably, the collector reinforcement should be developed over the entire length of the wall [Paulay, 1982]. An abrupt termination of collector reinforcement may result in localized failure and loss of anchorage. The reduction factor, ϕ, for chords and collectors should be taken as 0.60 to increase the likelihood that the diaphragm will remain elastic.

Axially Loaded Shear Walls. Any shear wall supporting axial loads must be checked against UBC §1921.6.5.4.

C407.1 Reinforcement

C407.1.1 ***UBC §1921.6.2.2 Walls With Single Curtains of Reinforcement.*** ACI §21.6.2.2. permits the use of shear walls with one curtain of steel wherever the in-plane factored shear force assigned to the wall does not exceed $2A_{cv}\sqrt{f'_c}$. However, no provisions are presented to address the special design considerations associated with such walls. In order to design ductile axially loaded single curtain walls, it is necessary to understand their behavior and to proportion and detail them to prevent brittle failure modes. Particular care is required in the design of these walls because once the flexural tensile yield strength of the wall reinforcement is exceeded, there is little inelastic reserve to fall back on. Lateral deflections increase rapidly at this stage, and if the axial load is very large, collapse may result.

Extensive testing has been carried out to study the behavior of axially loaded single curtain walls [ACI-SEAOC, 1982] and the engineer is directed to this reference for guidance. The detailing prescribed for the edges of shear walls is in recognition of the need to restrain the longitudinal reinforcement from buckling under extreme loading conditions and to develop the tensile strength of the shear reinforcement close to the edge of the wall.

C407.1.2 ***UBC §1921.6.2.4 Splices.*** Staggering of splices is recommended to avoid weak sections. The stagger requirement was deleted in the 1996 UBC Supplement.

C407.2 Design of Shear Walls for Flexural and Axial Loads

The 1994 UBC includes new provisions for shear walls that have not yet been adopted by ACI. These new provisions incorporate rational procedures to account for total flexural strength and section ductility, offering more reliable seismic performance than the previous empirical procedures.

Shear walls are employed in the following lateral force resisting systems:

1. As the bearing walls of a bearing wall system.
2. As walls that may be either separate from or integral with selected bays of a building frame system or a dual system.

These walls must have the strength to resist the shear, axial load, and moment due to:

1 $1.4D + 1.7L$

2. $1.4(D + L) + 1.4E$

3. $0.9D - 1.4E$

Further, to provide energy dissipation capacity, these provisions require transverse confinement reinforcement in the portions or zones of the wall sections that have concrete compressive strains in excess of 0.003 (in./in.) due to the inelastic deformation response of the structure when subjected to the design basis ground motion.

C407.2.1 ***UBC §1921.6.5.1 Analysis and Design of Wall Sections.*** The entire wall section may be designed as a column section to resist the required load combinations in section C407.2, Items 1, 2 and 3, above. The designer is free to select the distribution of the vertical reinforcement to provide an efficient and constructible section. Whatever distribution is selected, all properly developed vertical reinforcing steel to be placed within the wall section shall be included in the assessment of wall flexural strength used for purposes of determining shear demand and shall be included when assessing boundary zone confinement needs. Where intersecting walls or flanges form L-, T-, C-, or other shapes, the effect of these flanges shall also be considered. For a discussion of effective flange width, see Section C407.2.2. All vertical reinforcement included in the analysis should be adequately developed in tension. When a wall is supported on deep foundation elements, a load path should be established to ensure that the web and flange vertical reinforcement is effective and that the assumed reinforcement yield forces will be developed.

Wall elements that have the proportions corresponding to wall piers are also subject to the applicable provisions of UBC §1921.6.12.

Building frame systems and dual systems usually have shear walls constructed within selected bays of the required space frame. The columns of the space frame that are included in the wall section shall be considered as part of the wall section that resists seismic load combinations.

For the case of a system where the shear wall section includes reinforced concrete columns of the required SMRF, the column confinement requirements may be detailed using UBC §1921.6.5.6-2, without increase of hoop bar diameter. This is to avoid congestion caused by wall horizontal reinforcing anchored in the confined zone. The increase in spacing of confinement is allowed because the columns are an integral part of the wall and are less prone to the concentrated inelastic flexural and axial load deterioration effects that could occur in the joint regions of the open beam-column frame.

C407.2.2 ***UBC §1921.6.5.2 Effective Flange Width.*** Limits are necessary to define the effective flange width in flanged wall sections. The recommended 15 percent of wall height limit is larger than the 10 percent limit in the 1994 UBC, and has been proposed for adoption into the 1997 UBC. Even with this increase, it is a lower bound estimate. In some cases, a higher, upper bound estimate of effective flange width is needed. For example, in the determination of wall shear force present at probable flexural strength, or in the assessment of boundary zone confinement needs, a larger flange width should be assumed (Paulay and Priestly [1992] and Thomsen and Wallace [1995]). When assessing the need for boundary zone confinement, a larger effective flange width, such as 25 percent of the wall height on each side of the wall web is recommended. If there is a large concentration of reinforcement within a flange, consideration should be given to extending the effective width to include the extra reinforcement. The actual effective flange width is dependent on the deformation of the wall (Thomsen and Wallace [1995]). As the lateral deformation is increased, the effective flange width also increases.

C407.2.3 ***UBC §1921.6.5.3 Walls With Excessive Axial Loads.*** The limit of $0.35P_O$ is a conservative (lower bound) approximation of the "balanced point" on the column interaction curve below which the tensile steel strains reach yield prior to concrete compressive strains reaching 0.003. When axial loads exceed this limit, the more brittle concrete compression failure mode would govern; confinement similar to that required for columns would be necessary to provide some limited deformation capacity. For this Provision, P_u is the maximum compression value from the load combinations 1, 2, and 3 and should include forces from other yielding members (see Commentary Section C407.2.5) that may contribute to total axial load.

C407.2.4 ***UBC §1921.6.5.4 Design Method Triggers.*** The three listed triggers found in UBC §1921.6.5.4, Items 1, 2, and 3, are simple to evaluate for a wall or portion thereof, and they have been formulated to replace the previous gross section stress trigger of $0.2f_c'$ for boundary zones. They are intended to provide for a more realistic set of conditions to approximately determine when the wall section will not suffer strains beyond 0.003 when subjected to the design earthquake. The lower limit of $P_u \leq 0.05A_g f_c$ for nonsymmetrical wall sections is to allow for the large neutral axis depth and corresponding high compressive strains that can occur in the stems of these sections. A symmetrical wall with a limit of $P_u \le 10A_g f_c$ shall also have symmetrically distributed reinforcement. It is anticipated that most walls in low-rise structures will conform to the given trigger limits, in which case no further analysis is required. With respect to trigger 3, it is recommended that a limit on the shear span ratio $M_u / V_u l_w \le 3$ should apply when V_u is near to its limit. This modification has been adopted in the 1995 UBC Supplement. In general, caution should be taken when using the triggers for nonsymmetrical walls with large compression flanges or steel contents on one end of the wall.

For wall sections not meeting the trigger limits, the last paragraph of this Provision specifies the length of the confined boundary zone that would make any further analysis of strain conditions unnecessary. It is intended for cases where there are only a few walls that do not meet the trigger limits and where a detailed analysis is not justifiable in terms of any small savings in confinement steel. The boundary zone lengths vary with the intensity of axial load and have been formulated from parametric studies of neutral axis depths for symmetrical wall sections and may not be appropriate for the stems of nonsymmetrical sections.

C407.2.5 UBC §1921.6.5.5 General Methodology for Shear Wall Boundary Zone Analysis. The intent of this subsection is to allow and encourage the use of a range of detailed analyses to evaluate the intensity and extent of concrete compression strains in wall sections subject to maximum expected earthquake deformations. These detailed analyses require a procedure for the evaluation of the global deformation of the inelastic structure, and a subsequent analysis to find the local rotation and strain conditions at a particular wall section. From the experience and results that will be gained from these analyses, it will be possible to improve upon the limits and parameters of the simplified procedures for future Requirements. This general concept of deformation analysis should be extended to apply to the design and detailing of other structural systems such as frames and braced frames.

The maximum confined concrete strain limits given by Eqn. 21-8 in UBC §1921.6.5.5 as a function of R_w is to provide conservatism for walls of bearing wall systems without backup frames, and to encourage (or at least not discourage) the use of dual systems with their very desirable backup frames. The upper strain limit of 0.015 is established from research results for confined concrete [Park and Paulay, 1975].

In using these provisions, special considerations include:

1. It is to be noted that the UBC does not require that wall shear strength exceed the shear force present at the development of flexural strength. It is strongly recommended that engineers provide shear strength *in excess of* the shear corresponding to development of the wall flexural strength. This design approach is consistent with the underlying assumption used in the development of R_w values, which is that structural walls are capable of dissipating energy through inelastic displacement without substantial reduction of strength. If the wall shear strength is exhausted before the stable flexural strength is attained,

the wall ductility is greatly reduced, strength degradation will ensue and the amount of energy dissipated will be severely limited in comparison to a wall with a flexurally controlled yield mechanism.

2. These Requirements make repeated reference to the wall axial load, P_u', defined as $1.2D+0.5L+E$. Because, as explained below, the wall axial load can dramatically affect its available ductility, the quantity E must be carefully computed and should include all possible sources of axial load contribution. For example, a simple multistory cantilever wall supporting nonseismic beams may have zero axial load computed in the design seismic force analysis, but the vertical shears developed by yielding of the incoming beams can result in a net vertical load on the wall (Aktan and Bertero, 1983). Also, a coupled shear wall will develop an axial load based on the vertical shears present in the yielding coupling beams. The axial force should be determined on the basis of possible contributing effects in yielding elements of the system, as is currently required for steel structures such as Eccentrically Braced Frames.

General Strain Based Approach Background. UBC §1921.6.5.5 defines compression strain limits beyond which the boundary zone detail requirements of UBC §1921.6.5.6 shall be provided and also provides a methodology for estimating these strains. This methodology is described below, following a general discussion of this new strain-based approach.

In general terms, if flexural compression strains exceed the limiting value when the wall is displaced to its maximum inelastic deformation during the design earthquake, confinement of the compression or boundary zone is required. These Requirements require confinement of the boundary zone wherever compression strains in the wall critical section (at the plastic hinge) exceed a strain level of 0.003. In any concrete section yielding in flexure, a rapid loss of strength occurs once the unconfined concrete begins to spall, see Figures C407-1 and C407-2. Tests show that spalling of unconfined concrete initiates at strains on the order of 0.004 inch/inch; the 0.003 strain limit is thus conservative. In older versions of the UBC, the "trigger" for boundary zone confinement was based on combined gross section stresses; this now defunct procedure was found to be in some cases unconservative and in others overly conservative.

Many parameters influence the magnitude of compression strain developed in a wall (or in any flexural member). All other factors being equal, higher axial load, higher flexural steel content and larger height-to-width ratios will yield higher compression strains for the same total wall displacement at the top of the wall. For example, a concrete wall without adequate confinement and with an axial load higher than the balanced load will crush (develop strains in excess of 0.003) before the tensile reinforcement yields, and has, by definition, no ductility. Also, a T-section wall will have higher compression strains when the flange is in tension than when the flange is in compression, due to the higher steel content present in the flange of the wall.

The ductility available in a wall can be related to the neutral axis depth at ultimate strength. A large neutral axis depth (which implies that a large area in compression is needed to offset either a high axial load or a high steel content) is indicative of possibly low available ductility. In the New Zealand Building Code, boundary zone confinement triggers are based on the neutral axis depth. If the neutral axis depth exceeds a critical value [Paulay and Priestly, 1992], boundary zone confinement is required. Additional references on similar approaches to shear wall design include: Clarkson University [1992], Wallace [1992], and Wood [1991].

Compression strains are determined using one of two possible approaches, denoted as items 1 and 2 in UBC §1921.6.5.5.

Item 1 General Approach. Item 1 of UBC §1921.6.5.5 describes a generalized approach for the computation of concrete compressive strains at the maximum nonlinear displacement. Section C407.2.7 below describes an approximate approach for cantilever walls. Other types of walls require the generalized approach described in this section. These other types of walls would include:

1. Coupled Shear Walls, because even though hinging may occur at the base, the moment gradient and associated curvature-displacement relations will be much different from those for a cantilever wall.
2. Wall Piers, where moments reverse and plastic hinges form at the top and bottom of the pier.
3. Stepped Walls, where plastic hinging can occur at more than one elevation.
4. Walls where plastic hinging occurs at more than one location over the height of the wall or where plastic hinging occurs in a location other than the base of the wall.
5. Walls with nonsymmetrical cross-sections, such as T-, C-, or L-shaped walls, unless c_u' is calculated using first principles rather than empirical or cookbook equations.

This generalized approach requires the following analyses:

1. A rational design selection of the plastic hinge locations.
2. The establishment of nonlinear moment-curvature relationships for each critical section expected to form plastic hinges, utilizing appropriate levels of axial force in the moment-curvature analysis (this method of analysis is described in Park and Paulay [1975]). An example moment-curvature diagram is shown in Figure C407-2.
3. A plastic or inelastic analysis such as a static "pushover" analysis to 1) verify the plastic hinge location, to 2) account for the redistribution of forces and deformations of the wall system, to 3) determine the magnitude of plastic hinge rotations at each plastic hinge location associated with a given total wall displacement, and 4) to ensure that a brittle shear mechanism does not form.
4. Conversion of the plastic hinge rotation demands to section curvature demands, using kinematic relations derived for the specific mechanism geometry. (UBC

Equation 21-9 is a kinematic relation for the case of a cantilevered wall.) This analysis requires an estimate of the length of the plastic hinge region.

5. Conversion of section curvature demands to compressive strains. This information is obtained directly from the moment-curvature analysis of item 2 above.

This generalized approach is applied, by way of example, to the simple case of a cantilever wall system, as explained in section C407.2.7 below and illustrated on Figures C407-1 through C407-4.

Item 2 Approximate Approach. Item 2 of UBC §1921.6.5.5 describes an approximate approach for the computation of concrete compressive strains at the maximum nonlinear displacement for cantilever walls with symmetrical cross-sections. To use this simplified approach, the following wall characteristics need to be met:

1. The wall must be a single cantilevered element extending from the foundation level to the top of the building.
2. The wall can only form a plastic hinge at its "base" (the base need not be at the foundation level, but should be the elevation below which no plastic hinges form). This requires that all sections above the wall base must not yield as the base yields. The wall outside of the plastic hinge region should be designed for strengths corresponding to the moment and shear envelope present when the probable moment M_{pr} is developed at the plastic hinge. This will help eliminate the possibility of the plastic hinge forming in a location other than the base of the wall, possibly in a region that is not provided with boundary zone confinement.
3. The wall section expected to yield must have symmetry of strength and stiffness about the axis of bending (this need not apply if the engineer determines the neutral axis depth, c_u', using first principles rather than empirical or cookbook equations);
4. Although the UBC does not require it, it is strongly recommended that the wall shear strength be greater than the shear developed when the critical section has developed its *probable strength,* M_{pr}, *determined using* P_u' *and including all vertical reinforcement likely to contribute to flexural strength.* If shear failure precedes flexural yield, wall ductility is greatly reduced, and this simplified method of analysis will not accurately predict the wall compressive strains.

For walls that comply with the above, the simplified approximate approach proceeds as follows:

1. Establish the top of wall displacement (Δ_s) caused by the design seismic forces. This requires the use of the effective moment of inertia for the wall section over the full height of the wall. The effective moment of inertia can be approximated using the relation $I_{eff}/I_g = 14.5/F_y + P_u A_g f_c'$ [Paulay and Priestly, 1992].

However, this relation can yield unrealistically low ratios of I_{eff}/I_g. The effective moment of inertia may more accurately be computed using the approach given in ACI 318 [1995]; the effective moment of inertia concept is illustrated in Figure C407-1.

2. Establish the maximum displacement, Δ_m. This displacement is defined as R_w times the design level displacement, Δ_s. The 1994 UBC permitted Δ_m to be defined as $3(R_w/8)$ times the design level displacement. This was later revised to $\Delta_m = R_w \Delta_s$ to give a more realistic estimate of actual displacements. This revision is expected to be adopted in the 1997 UBC.
3. Establish the yield displacement, Δ_y at the top of the wall. The approximation permitted in the Requirements is valid only so long as Δ_s is computed using the effective moment of inertia.
4. Establish the inelastic portion of the wall displacement, $\Delta_i = \Delta_m - \Delta_y$. This quantity is used directly in UBC equation 21-9. The displacements Δ_y, Δ_i and Δ_m are illustrated on Figure C407-4b. Derivations of kinematic relations for Δ_y and ϕ_i for the case of a cantilever wall are defined in Figure C407-4.
5. Establish the section curvature at first yield, ϕ_y. The yield curvature, ϕ_y, is defined in Figure C407-3a. This can be approximated as $= 0.003/l_w$, which is reasonably accurate for most wall sections [Paulay and Priestly, 1992]. The section curvature at yield can also be determined by a moment-curvature analysis of the section.
6. Estimate the length of the plastic hinge. The vertical length of plastic hinge is illustrated on Figure C407-4a. If the plastic hinge length is underestimated, the total curvature ϕ_t will be overestimated, as will be the peak compression strain.

   If the plastic hinge length is overestimated, the total curvature ϕ_t and the peak compression strain will be underestimated. As the ultimate goal of this analysis is to determine if confinement of portions of the wall is needed on the basis of compression strain, it is conservative to underestimate the plastic hinge length. Available research data on plastic hinge length for walls is sparse and caution is warranted. The UBC permits the use of $l_p = 0.50 l_w$, which is reasonable for truly flexural walls, but may overestimate the length for long, squat walls.
7. Solve for the total inelastic section curvature, ϕ_t. This is found using equation 21-9. This relation, derived on Figure C407-4d, is a relationship between curvature and displacement, and is derived using first principles and the moment area theorem [Park and Paulay, 1975].

8. Establish the depth to neutral axis, c'_u, corresponding to the top of wall displacement, Δ_m. This neutral axis depth is illustrated on Figure C407-3c. The actual value of the neutral axis depth corresponding to the displacement, Δ_m, may be determined by a moment-curvature analysis of the section, or an approximate value may be computed as follows: The neutral axis depth, c'_u, may be computed as the neutral axis depth at the development of the nominal flexural strength of the wall, as defined by Figure C407-3b. This requires a strain compatibility analysis of the section using the very same principles outlined in UBC §1910.2 for the general case of axial-flexure interaction of concrete sections. For this analysis, the axial load shall be taken as $P'_u = 1.2D+0.5L+\mathrm{E}$ and all properly anchored, spliced, and developed vertical reinforcement (such as vertical web reinforcement) shall be included and shall be assumed as yielded in tension or compression. This latter assumption is accurate for cyclic loading because the vertical reinforcement in the compression zone will have yielded in tension in a previous cycle, and must yield in compression before the tensile cracks close and allow the concrete to carry compression. Similarly, tensile reinforcement near the neutral axis is likely to still be carrying yield-level forces from previous cycles.
9. Solve for the compressive strain associated with the total inelastic section curvature, ϕ_t. From the first principles and the definition of curvature, this is determined as the product of ϕ_t and c'_u, and is illustrated on Figure C407-3c.
10. Finally, determine the length over which boundary zone confinement must be provided. Figure C407-3c illustrates the horizontal extent of this zone, which is determined using the geometry of the strain diagram.

This approach provides a reasonable estimate of concrete compressive strains for simple cantilever walls. Other wall types should be analyzed using the general approach discussed in Section C407.2.6 above. The designer is cautioned to use care in establishing the axial load, P'_u, and the plastic hinge length, l_p; actual compression strains are very sensitive to these parameters. Designers are further cautioned against *neglecting* wall web vertical reinforcement in the calculations; over-reinforced walls can be quite brittle, as illustrated on Figure C407-2. Neglecting vertical web reinforcement is: 1) not necessarily conservative because increased flexural strength can increase the likelihood of a wall failing in shear with little ductility; and 2) web reinforcement intentionally designed as flexural reinforcement can substantially reduce congestion in the wall boundary zones as well as improve overall wall ductility and displacement capacity.

These new provisions provide a rational method of analysis to determine if boundary zone confinement is needed. The intent of these new provisions was to relax the previously onerous gross sectional stress based requirements which, more often than not, led to concrete placement problems and economic penalty, both with little or no increase in safety. However, designers are encouraged to consider the use of boundary zone confinement in the expected plastic hinge regions of truly flexural walls even when not required by these new provisions, to offset uncertainty associated with the estimate of maximum wall displacement and to minimize buckling of vertical reinforcement yielding in compression. The associated cost premium is more than offset by added safety, ductility, and reduced postearthquake repairs.

C407.2.6 UBC §1921.6.5.6 Shear Wall Boundary Zone Detail Requirements. This section establishes minimum dimensional and reinforcement detailing requirements for assuring acceptable performance.

Item 1. Gives the minimum boundary zone thickness requirement of $l_u/16$. This is a decrease from the 1994 UBC requirement of $l_u/10$ and is expected to be adopted in the 1997 UBC. It is strongly recommended that the wall boundary thickness limit of $l_u/16$ be applied at all potential plastic hinge locations, regardless of whether boundary zone confinement is required. This value is provided to preclude out-of-plane buckling failure of the wall compression zone. The limit is tentative and is subject to change as more research and experience becomes available. Control of buckling failure is more critical with the boundary Requirements specified herein than was necessary with previous Requirements, where boundary elements were required to carry all vertical loads and therefore were larger and contained significantly more cross-section and vertical reinforcing. Because of the reduced amount of steel required in these present Requirements, the level of tensile strain in the boundary zone steel can be large due to reversed flexural loading. Research described in Paulay and Priestly [1993] shows that wall buckling is aggravated when there are high residual tensile strains in the steel and the associated boundary zone is subjected to compression in successive flexural load cycles.

Items 1. 2 and 1.3. Minimum horizontal and vertical boundary zone dimensions have been defined to provide confinement within the potential plastic hinge regions.

Item 1.4. Specific requirements for defining effective flange participation in the boundary zone have been provided. See Commentary C407.2.2.

Item 2. Eqn. 21-10 is essentially the same as that for columns in special moment resisting frames. However, for boundary zones, h_c is the width of the confined core measured normal to the plane of the wall at the location of the hoop.

The spacing of the smaller of 6 inches or 6 bar diameters is sufficient for the prevention of inelastic buckling of longitudinal steel and is large enough to provide for constructibility. All vertical bars on the perimeter should have horizontal steel passing around them. Nonoverlapping ties are not allowed, since there could be some bars with only cross-tie confinement in large boundary zone configurations.

It is strongly recommended that all wall edges in potential plastic hinge regions have ties spaced at $6d_b$ or 6 inches maximum, to limit buckling of bars yielding in tension and/or compression [Paulay and Priestly, 1992].

Item 4. A minimum area of vertical reinforcement is provided to control the potential for excessive elongation of the boundary zone flexural reinforcement in lightly reinforced wall sections.

C407.3 UBC §1921.6.6.3 Boundaries of Structural Diaphragms

UBC §1921.6.6.3 was added to the 1996 UBC Supplement in response to diaphragm failures observed in Northridge following the 1994 earthquake. The intent of these Requirements is to provide some level of confinement and cover for chord and collector reinforcement. In Northridge, it was observed that large diameter chord and collector bars and/or closely spaced lap splices tended to break away the limited concrete cover, especially in concrete topped precast slab systems. It was apparent that the reinforcement so exposed underwent large, possibly inelastic, displacement levels. Once the concrete cover spalled, the chord/collector bars were rendered ineffective, and in some cases, catastrophic building failures ensued.

C407.4 UBC §1921.6.8 Discontinuous Walls

This cross-reference has been added to flag the special detailing requirements for columns supporting discontinuous wall elements.

C407.5 Coupling Beams

The entire section on the design of coupling beams has been replaced. The revised Requirements are modeled after provisions adopted into the 1994 edition of NEHRP and thus represent a more national consensus. A coupling beam is defined as a horizontal element in plane with and connecting two shear walls.

C407.5.1 UBC §1921.6.9.1 General Requirements for Coupling Beams Responding Primarily in Flexure. When the clear span to depth ratio is greater than or equal to 4, the coupling beams are designed in accordance with the general requirements of UBC §1921.2 and in conformance with the requirements of UBC §§1921.2 and 1921.3. The designer may, if substantiated by a rational analysis, waive the requirements of UBC §§1921.3.1.3 and 1921.3.1.4 if it can be shown that lateral stability is adequate or if alternate means of maintaining lateral stability is provided. It is implied that the structure is analyzed without the coupling beams and the strength, stiffness, and stability of the structure is ensured.

C407.5.2 UBC §1921.6.9.2 General Requirements for Coupling Beams Responding Primarily in Shear. Tests indicate that short flexural members with small clear-span-to-depth ratios behave differently than slender flexural members. When short coupling beams were subjected to inelastic load cycles, large flexural cracks formed at both beam ends. With increasing load reversals, cracks from each direction of loading interconnected, forming a vertical plane of weakness at each end of the beam. Thus, instead of a conventional truss mechanism (as assumed for the shear strength of slender beams), shear transfer across the plane of weakness was provided primarily by aggregate interlock and shear friction. Under subsequent inelastic load cycles, the shear resisting mechanism deteriorated rapidly, resulting in a loss of capacity by sliding shear.

It has been found that increasing the number of hoops confining is not effective in improving resistance against sliding shear. Therefore, various configurations of special shear reinforcement to improve seismic performance of short coupling beams were tested. Full-length diagonal reinforcement was found to produce the greatest energy dissipation and deformation capacities.

In tests conducted by the Portland Cement Association (PCA) [Barney, 1978], beam specimens with a clear-span-to-depth ratio of 2.8 were able to sustain over 34 reversing load cycles. A maximum nominal shear stress of $12.5\sqrt{f_c'}$ was recorded and a maximum imposed deformation of over nine times the yield deflection was measured. In addition, the specimen was able to dissipate three times more energy than a comparable specimen without diagonal reinforcement. No sign of sliding shear was observed at completion of testing. In this and research by others, diagonal shear reinforcement was designed by the proposed formula to resist the total shear force by truss action.

Based on test results, Park and Paulay [1975] recommended that diagonal shear reinforcement be used to carry 75 percent of the induced shear in flexural members when nominal shear stresses under load reversals are larger than $3\sqrt{f_c'}$ psi. When nominal shear stress exceeds $4.5\sqrt{f_c'}$ psi, Paulay [1992] recommended that diagonal reinforcement should carry 100 percent of the induced cyclic shear forces.

For clear-span-to-depth ratios greater than 4.0, PCA tests indicated that specimens with conventional reinforcement resisted over 46 inelastic reversing load cycles. At the same time, maximum deformation of 13 times yield deflection was measured. Therefore, experimental data have shown that beams with a clear-span-to-depth ratio greater than 4.0 do not require diagonal reinforcement. In addition, diagonal reinforcement is not very effective in beams with clear-span-to-depth ratios greater than 4.0.

It is recommended that pierced walls with weak piers relative to the coupling beams be avoided. For such walls, very large inelastic deformation demands may be concentrated on a single floor level. When such a mechanism forms, the portion of the wall above the mechanism will deform primarily as a rigid body and the inelastic demands imposed on the piers within the mechanism may not be properly predicted by the static force method.

C407.5.3 ***UBC §1921.6.9.3 Coupling Beams, Detail Requirements for Diagonal Bars.*** The requirement for transverse reinforcement, recommended by Paulay [1992], is to ensure the ability of the diagonal bar cage to sustain yield level compression forces. Without the transverse confinement required, the bars would buckle in compression and lose load-carrying capability.

Additional detailing considerations are suggested below:

1. The development length of the diagonal reinforcement, according to Paulay [1992], should be 1.5 times the nominal development length. Also, the drawings should clearly specify from what point the development length is measured.
2. The force generated in the diagonal reinforcement must be distributed into the body of the wall at each end. Terminating the diagonal bars in a lightly reinforced wall is not to be encouraged. A confined cage of vertical longitudinal reinforcement is recommended to act as a collector of forces transmitted by the coupling beam into the main body of the wall.
3. Consideration should be given to the probable shear force to be developed. If substantial or inadvertent flexural reinforcement occurs in the coupling beam, the shear force to be induced in that member may be larger than predicted by static force analysis.
4. Consideration must be given to constructibility of diagonally reinforced coupling beams. The wall thickness may be controlled by the layering of reinforcement within and through the coupling beam. A means of placing concrete in congested coupling beams must be provided.
5. Where coupling beams are located on the exterior of a building and are clad with architectural finishes, the deformation compatibility of the structure and finish deserves special attention. The shearing deformations of the coupling beams will be primarily vertical, as in link beams of eccentrically braced frames. Special jointing of the finish may be required to comply with deformation compatibility requirements of UBC §1631.

C407.5.4 UBC §1921.6.9.4 Coupling Beams, Minimum Reinforcement. The reference to UBC §1910.5 (minimum reinforcement for flexural members) may result in potentially excessive longitudinal reinforcement and reinforcement congestion for coupling beams with diagonal reinforcement. In tests by Park and Paulay [1975], coupling beams with only light secondary top and bottom longitudinal reinforcement performed satisfactorily; in fact, the top and bottom longitudinal bars did not comply with Section 1910.5 and were neither anchored nor continuous bars.

C407.6 UBC §1921.6.10 Toppings on Precast Diaphragms

Although ACI 318 Chapter 21 did not address the use of precast elements in floor diaphragms until the 1995 edition, this provision establishes a method of using such elements for the gravity load-carrying system, while maintaining the benefits of monolithic construction for seismic resistance.

C407.7 UBC §1921.6.11 Minimum Thickness of Diaphragms

With minimum reinforcement requirements based on concrete area (ACI 7.12), there is some incentive in design to save both concrete and steel by using thin diaphragms. Minimum diaphragm thicknesses are specified to ensure a minimum level of performance in all diaphragms.

The requirements for topping slab thickness over precast floor and roof systems were revised in response to damage observed in the 1994 Northridge earthquake. The minimum thickness was increased to 3 inches for this type of system, and further, to ensure adequate thickness for lap splices, the minimum thickness is also controlled by the $6d_b$ requirement.

It is further recommended that at least 2.5 d_b cover be provided over reinforcement and that the use of prestress steel for chords and collectors be discouraged.

C407.8 UBC §1921.6.12 Wall Piers

Current ACI-318 [1988] code provisions have no lower bound limit on the length of a shear wall, and do not adequately cover the design and detailing of slender wall segments. The observed earthquake damage due to short column effects in narrow wall piers shows the need for adequate transverse reinforcement.

The special transverse reinforcement in wall piers is not required when:

1. Wall piers and spandrels are designed as specially reinforced members meeting the requirements of UBC §1921.4 and 1921.3.
2. Wall piers are not part of the lateral force resisting system, and conform to the deformation compatibility requirements under UBC §1921.7.
3. The stiffness of longer wall segments is sufficiently greater than the combined stiffnesses of the wall piers. ACI 318-83 commentary §10.11.2 suggests a stiffness factor of 6 for a column to be considered braced. This relative stiffness ratio has been used for wall piers.

The design shear force V_e should be determined by considering the probable flexural strength at the ends of a wall pier based on stress in the tensile reinforcement of $1.25 f_y$ and a strength reduction factor equal to 1, and then dividing by the clear height of the opening. Wall piers supporting deep spandrels are similar to columns supporting discontinuous shear walls. Thus, the requirements for the extent and spacing of transverse reinforcement are similar to those under UBC §1921.4.4.5.

C408 UBC §1921.7 Frame Members Not Part of the Lateral Force Resisting System

In conjunction with the revision of UBC §1631, Deformation Compatibility Requirements, and in response to damage observed in the 1994 Northridge earthquake, UBC §1921.7 was completely revised to better ensure that beams and columns that are not designed to resist design seismic forces can accommodate expected seismic deformations. The following Commentary refers to the sections in the revised version given in Requirements Section 402.13.

This new procedure simplifies compliance with code-specified deformation compatibility requirements. The process of compliance is outlined as follows:

1. Analyze the structure in conformance with UBC §1631.2.4.
2. Compare the induced shears and moments combined with factored gravity loads with the corresponding member strengths. Two possible scenarios are addressed:

a. When induced forces combined with the factored gravity load effects *do not exceed* design strengths, the detailing provisions of UBC §1921.7.2 are applicable. Those requirements depend on the relative level of axial load in the member.

b. When induced forces combined with the factored gravity load effects *exceed* the design strengths, the detailing provisions of UBC §1921.7.3 are applicable. In this case, inelastic response of the member will occur. Therefore, more stringent detail and material requirements are invoked. Again, these requirements depend on the relative level of axial load in the member.

3. It is recognized in UBC §1921.7.1 that computation of induced forces and deformations is both time consuming and imprecise. Therefore, the designer is permitted to not make the calculations *provided the more stringent detail requirements of UBC §1921.7.3 are adhered to.*

C408.1 Factored Gravity Forces

The factored gravity load effects are to be combined with 1.0 times the effects of the seismic forces induced by the deformation specified in UBC §§1921.7.

C408.1.1 ***UBC §1921.7.2.1 Members Not Exceeding Design Strength and With Low Axial Loads.*** Members that respond essentially in flexure are subject to the minimum and maximum reinforcement requirements for flexural members of frames. Also, the maximum spacing of stirrups is reduced to $d/2$ throughout the member length.

C408.1.2 ***UBC §1921.7.2.2 Members Not Exceeding Design Strength and With Moderate Axial Loads.*** When the factored axial load exceeds $(A_g f_c'/10)$, portions of the requirements for framing members subjected to bending and axial load are applicable. Closely spaced transverse reinforcement throughout the member length and designing for the maximum shear that can be developed are two new major requirements. All requirements in UBC §1921.4 are not invoked, because it is presumed that the member will not undergo inelastic action. However, attention to minimum reinforcement and transverse reinforcement detailing is required to ensure a minimum amount of ductility, as it is recognized that the actual displacement may exceed the $3R_w/8$ value assumed.

C408.1.3 ***UBC §1921.7.2.3 Members Not Exceeding Design Strength and With High Axial Loads.*** The axial load level, $0.3P_o$, is approximately the axial load level at the balanced condition. At axial load levels of this magnitude, the deformation capacity of beam-column members in the elastic range is extremely limited. Inelastic deformation capacity is also limited; it is possible to achieve the ultimate strength prior to yielding of the member. For this reason, the complete transverse reinforcing requirements of UBC §1921.4.4 are applicable. This is different from the corresponding provision of ACI 318-95 where the limit is $0.35\ P_o$ and only one-half the transverse steel amount (required in a ductile column) is required.

C408.2 Members Exceeding Design Strength

When the induced forces exceed the design strengths, inelastic response of the member is more than likely.

C408.2.1 ***UBC §1921.7.3.1 Members Exceeding Design Strength Materials.*** Materials must comply with the same requirements as for frame members designed to resist seismic forces.

C408.2.2 ***UBC §1921.7.3.3 Members Exceeding Design Strength and With Moderate to High Axial Loads.*** The requirements are the same as UBC §1921.7.2.3, with the additional requirement of transverse reinforcement through the joint region.

C409 UBC §1921.8 Requirements for Frames in Seismic Zone 2

Use of unfactored tributary gravity loads in combination with the nominal beam strengths represents a rational approach for the calculation of the design shear force. See Commentary Section C403.4.1. for an expanded discussion.

C410 Constructibility

Conformance with the Requirements of this Chapter will not necessarily result in a satisfactory structure if reinforcing steel cannot be placed because of interference, and/or if concrete cannot be placed and consolidated because of restricted space and clutter. The following text is from a report by ACI Committee 309, *Guide for Consolidation of Concrete* [ACI-309, 1987].

C411 Design and Detailing Prerequisites

In designing structural members and detailing formwork and reinforcement, consideration should be given to depositing the freshly mixed concrete as closely as possible to its final position in such a way that segregation, honeycombing, and other surface and internal imperfections are minimized. Also, the method of consolidation should be carefully considered when detailing reinforcement and formwork. For example, for internal vibration, openings in the reinforcement must be provided to allow insertion of vibrators. Typically, 4 by 6-inch openings at 24-inch centers are used.

These items require that special attention be directed to member size, bar size, bar location, bar spacing, and other factors that influence the placing and consolidation of concrete. This is particularly true in structures designed for seismic loads, where the reinforcement often becomes extremely congested and effective concrete consolidation using conventional mixtures and procedures becomes impossible.

The engineer should communicate with the constructor during the early concrete phases. Problem areas should be recognized in time to take appropriate remedial measures such as staggering splices, grouping bars, modifying stirrup spacing, and increasing section size. When conditions contributing to substandard consolidation exist, one or more of the following actions should be taken: redesign the member, redesign the reinforcing steel, modify the mixture, utilize mock-up tests to develop a procedure, and alert the constructor to critical conditions.

In addition to the concrete placement capabilities, the engineer should consider the configuration and assembly of the complete reinforcing steel system. It is common construction practice to prefabricate bar cages for beams, columns, wall boundary elements, and shear wall curtains; and to assemble or join these elements in place. This procedure is often the most cost-effective, and results in an accurate and stable positioning of reinforcing steel. However, care and planning are necessary to properly connect the pre-tied elements. Hooks, cross-ties, ties, lap splices, and mechanical splices all need appropriate shapes and positioning to allow placement without interference.

References

ACI-309, 1987. ACI Committee 309, "Guide for Consolidation of Concrete," *ACI Materials Journal*, Vol. 84, no. 5.

ACI 318, 1995. Committee 318, "Building Code Requirements for Reinforced Concrete."

ACI-ASCE, 1988. Committee 352, "Recommendations for Design of Slab Column Connections in Monolithic Reinforced Concrete Structures," *ACI Structural Journal*, Vol. 85, no. 6.

ACI-SEAOC, 1982. American Concrete Institute Southern California Chapter and Structural Engineers Association of Southern California, *Report of the Task Committee on Slender Walls*, Los Angeles, California.

Aktan, A.E., V.V. Bertero, A.A. Chowdhury, and T. Nagashima, 1983. *Experimental and Analytical Predictions of the Mechanical Characteristics of a 1/5 Scale Model of a 7-Story R/C Frame-Wall Building Structure*, Report No. 83/13, EERC, University of California, Berkeley.

Aoyama, H., et al., 1992. "Development of Advanced R.C. Buildings with High Strength and High-Quality Materials," *Proceedings of Tenth World Conference on Earthquake Engineering*, held in Madrid, Spain, July 19-24. Balkema Publishers, Rotterdam, Holland.

ATC-11, 1983. Applied Technology Council, *Seismic Resistance of Reinforced Concrete Shear Walls and Frame Joints: Implications of Recent Research for Design Engineers*, Report No. ATC-11, Redwood City, California.

Barney, G. B., K.N. Shiu, B.G. Rabbat, A.E. Fiorato, H.G. Russell, and W.G. Corley, 1978. *Earthquake Resistant Structural Walls: Test of Coupling Beams*, Report to the National Science Foundation (RANN), Grant No. ENV74-14766, Construction Technology Laboratories, Skokie, Illinois.

Clarkson University, 1992. "Development of New Code Provisions for Seismic Design of Reinforced Concrete Shear Walls," Report No. CU/CEE-92/3, Dept. of Civil Engineering, Clarkson University, Potsdam, New York.

Ghosh, S.K. and C.A. Etraify, 1993. "Design of Reinforced Concrete Shear Walls in Regions of High Seismicity," PCA memo.

Hawkins, N.M., I. Lin, and T. Veda, 1987. "Anchorage of Reinforcing Bars for Seismic Forces," *ACI Structural Journal*, Vol. 84, no. 5.

Kaar, P.H., Hanson, N. W. and Capell, H. T., 1978. *Stress-Strain Characteristics of High-Strength Concrete*, Publication SP-55, American Concrete Institute, Detroit, Michigan.

Kabeyasawa, T., et al., 1992. "Tests and Analysis of Ultra-High Strength Reinforced Concrete Shear Walls," *Proceedings of Tenth World Conference on Earthquake Engineering*, held in Madrid, Spain, July 19-24. Balkema Publishers, Rotterdam, Holland.

Kaku, T., et al., 1991. "Bond Splitting Strength of Reinforced Concrete Beams with High-Strength Concrete," *Proceedings of Japan Concrete Institute*, Vol. 13, no. 2.

Miura, A. et al., 1990. "Shear Failure of R.C. Exterior Beam-Column Joints Composed of High Strength Materials," *Proceedings of the Architectural Institute of Japan Annual Meeting*, October, 1990.

Moehle, J.P., M.E. Kreger, and R. Leon, 1988. "Background to Recommendations for Design of Reinforced Concrete Slab-Column Connections," *ACI Structural Journal*, Vol. 85, no. 6.

Oesterle, R.G., A.E. Fiorato, and W.G. Corley, 1980. "Reinforcement Details for Earthquake-Resistant Structural Walls," Concrete International, Vol. 2, no. 12.

Okamura, H., et al., 1987. "Bond Characteristics in Post-Yield Range of Deformed Bars," Translation from *Proceedings of JSCE*, no. 378/V-6.

Park, R., and T. Paulay, 1975. *Reinforced Concrete Structures*, John Wiley and Sons.

Paulay, T., 1986. "A Critique of the Special Provisions for Seismic Design of the Building Code Requirements for Reinforced Concrete" ACI 318-83, *ACI Structural Journal*, Vol. 83, no. 2.

Paulay, T., 1986. "The Design of Ductile Reinforced Concrete Structural Walls for Earthquake Resistance," *Earthquake Spectra*, Vol. 2, no. 4.

Paulay, T., and M.J.N. Priestly, 1993. "Stability of Ductile Structural Walls," *ACI Structural Journal*, Vol. 90, no. 4.

Paulay, T., M.J.N. Priestly, and Synge, 1982. "Ductility in Earthquake Resisting Squat Shear Walls," ACI Structural Journal.

Paulay, T., and M.J.N. Priestly, 1992. "Seismic Design of Reinforced Concrete and Masonry Buildings," John Wiley & Sons, Inc.

Rabbat, B. G., J. I. Daniel, T. L. Weinmann, and T. L. Hanson, 1986. "Seismic Behavior of Lightweight and Normal Weight Concrete Column," ACI Journal, Proceedings V. 83, no. 1.

Sugano, T., et al., 1988. "Experimental Studies on Seismic Behavior of High Strength Concrete Columns Laterally Reinforced with High Strength Steel Bars," *Proceedings of Ninth World Conference on Earthquake Engineering*, held Aug. 2-9, 1988 in Tokyo, Japan, Vol. IV.

Wallace, John A., and J.P. Moehle, 1992. "Ductility and Detailing Requirements of Bearing Wall Buildings," *ASCE Structural Journal.*

Wood, Sharon L., 1991. "Performance of Reinforced Concrete Buildings During the 1985 Chile Earthquake: Implications for Design of Structural Walls," *Earthquake Spectra*, Vol. 7, no. 4.

Zsutty, T. C., 1985. "Empirical Study of Bar Development Data," *ASCE Journal of Structural Engineering*, Vol. 111, no. 1.

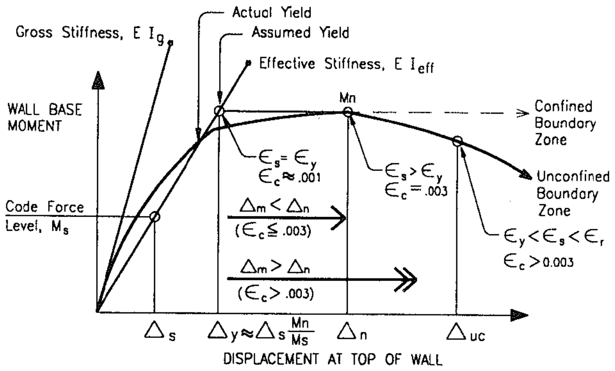


Figure C407-1. *Wall moment versus displacement*

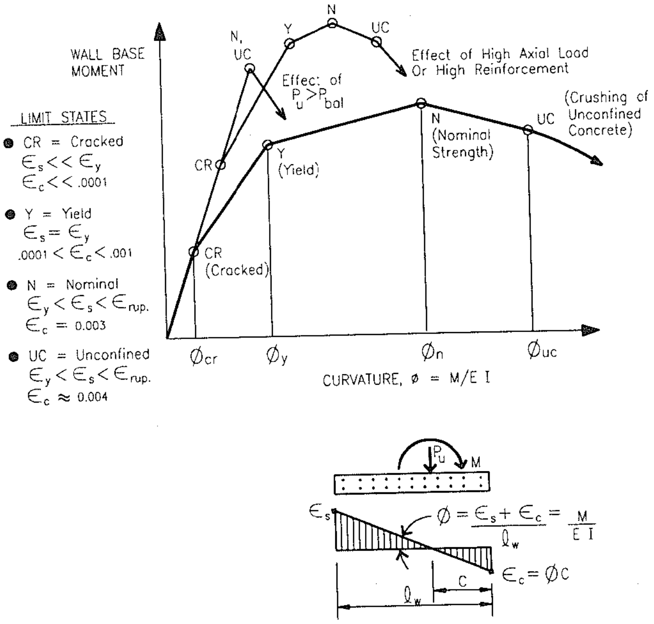

WALL BASE MOMENT
LIMIT STATES
● CR = Cracked
$\epsilon_s << \epsilon_y$
$\epsilon_c << .0001$
● Y = Yield
$\epsilon_s = \epsilon_y$
$.0001 < \epsilon_c < .001$
● N = Nominal
$\epsilon_y < \epsilon_s < \epsilon_{rup.}$
$\epsilon_c = 0.003$
● UC = Unconfined
$\epsilon_y < \epsilon_s < \epsilon_{rup.}$
$\epsilon_c \approx 0.004$
N, UC
Y
N
UC
Effect of High Axial Load Or High Reinforcement
Effect of $P_u > P_{bal}$
CR
Y (Yield)
N (Nominal Strength)
UC (Crushing of Unconfined Concrete)
CR (Cracked)
ϕ_{cr}
ϕ_y
ϕ_n
ϕ_{uc}
CURVATURE, ϕ = M/E I
P_u
M
ϵ_s
$\phi = \frac{\epsilon_s + \epsilon_c}{l_w} = \frac{M}{E\,I}$
$\epsilon_c = \phi c$
c
l_w

Figure C407-2. *Moment curvature diagrams*

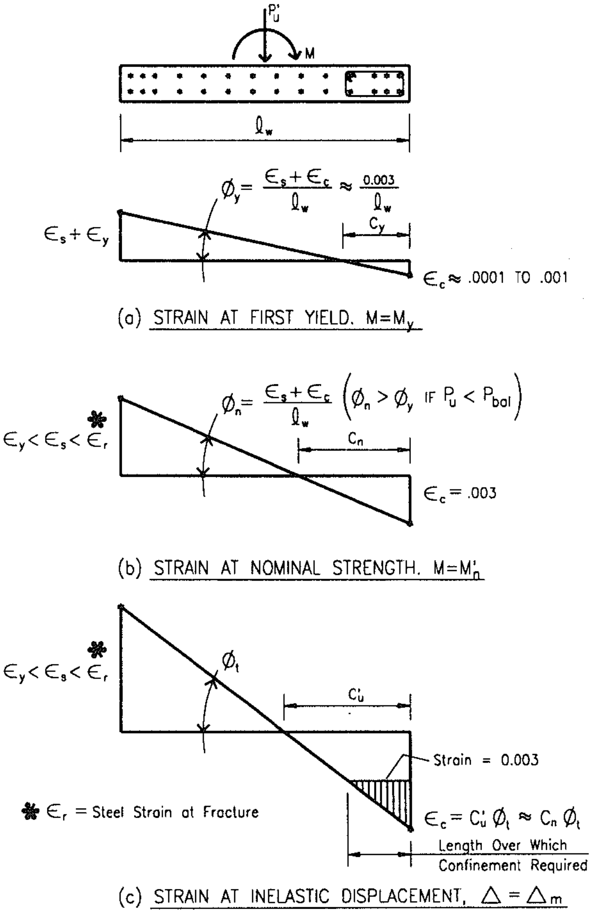
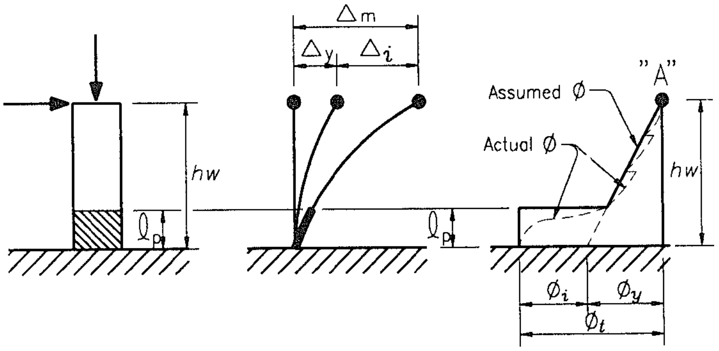

P_u'
M
ℓ_w
$\phi_y = \frac{\epsilon_s + \epsilon_c}{\ell_w} \approx \frac{0.003}{\ell_w}$
$\epsilon_s + \epsilon_y$
C_y
$\epsilon_c \approx .0001$ TO .001
(a) STRAIN AT FIRST YIELD. $M=M_y$
$\phi_n = \frac{\epsilon_s + \epsilon_c}{\ell_w}$ $\left(\phi_n > \phi_y \text{ IF } P_u < P_{bal}\right)$
✱ $\epsilon_y < \epsilon_s < \epsilon_r$
C_n
$\epsilon_c = .003$
(b) STRAIN AT NOMINAL STRENGTH. $M=M_n'$
✱ $\epsilon_y < \epsilon_s < \epsilon_r$
ϕ_t
C_u'
Strain = 0.003
✱ ϵ_r = Steel Strain at Fracture
$\epsilon_c = C_u' \phi_t \approx C_n \phi_t$
Length Over Which Confinement Required
(c) STRAIN AT INELASTIC DISPLACEMENT, $\Delta = \Delta_m$

Figure C407-3. *Strain compatibility diagrams*



(a) Cantilever Wall Under Simplified Loading

(b) Deflections

(c) Curvature At Maximum Response

AT FIRST YIELD:

$$\Delta_y = \left(1/2\,\phi_y\,hw\right)\left(2/3\,hw\right) = 1/3\,\phi_y\,hw^2$$

(By Moment-Area Theorem About "A")
(This relation may not be used for UBC approach)

AT MAXIMUM RESPONSE:

$$\Delta_m = \Delta_i + \Delta_y$$

$$\Delta_i \cong \left(\phi_i\,\ell_p\right)\left(hw - \ell_p/2\right)$$

(By Moment-Area Theorem About "A")

TO FIND ϕ_t:

$$\phi_t = \phi_y + \phi_i$$

$$\phi_t = \phi_y + \frac{\Delta_i}{\ell_p(hw - \ell_p/2)}$$

(d) Derivation Of Simple $\phi - \Delta$ Equation

Figure C407-4. *Approximate determination of* ϕ_t

CHAPTER 5
Lightweight Metals

C501 UBC §2001 General Requirements

Lightweight metals is a new chapter inserted to parallel the chapters of the 1994 UBC. This class of material has not been examined in detail and specific provisions have not yet been adopted for lateral force resistance. Research needs to be conducted to determine design procedures to ensure that strength and ductility demand requirements are not exceeded.

CHAPTER 6
Reinforced Masonry

C601 General Requirements

The provisions of the 1994 UBC and the additional Requirements of this chapter represent acceptable minimum requirements necessary to achieve the seismic design objectives of these Requirements.

Design methods using strength design have been added to the 1994 UBC and expanded to cover: shear walls and wall frame as lateral force-resisting systems; beams, piers and columns as components not part of the lateral force-resisting systems; anchorage; and reinforcement.

C602 Recommended Modifications to 1994 UBC

C602.1 Standards of Quality

Modify: ***UBC §2102.2, Item 10 Reinforcement.*** The use of plain bars in reinforced masonry construction has been prohibited, except for joint reinforcement, because of the poor bond capacity of plain bars, particularly under cyclic loading.

C602.2 General Design Regulations

C602.2.1 *Modify*: ***UBC §2106.1.12.4, Item 2.1 Shear Walls: Reinforcement.*** There is a lack of experience in actual earthquakes on the performance of masonry with joint reinforcement. Therefore, it was decided not to allow this type of reinforcement in the design for shear.

C602.2.2 *Modify*: ***UBC §2106.3.2 Plain Bars.*** The use of plain bars is prohibited because of their poor bond capacity.

C602.3 Design of Reinforced Masonry

C602.3.1 *Delete*: ***UBC §2107.2.2 Reinforcement.*** Working stress reinforcement provisions have been deleted and replaced with ultimate strength design provisions of UBC §2108 to provide improved postyield performance.

C602.3.2 *Delete:* ***UBC §2107.2.12 Lap Splice Increases.*** Working stress reinforcement provisions have been deleted and replaced with ultimate strength design provisions of UBC §2108 to provide improved postyield performance.

C602.4 Strength Design of Masonry

Modify: ***UBC §2108.2.2.6 Development.*** There should not be a cap on development length, as larger reinforcing bar sizes need very long development lengths to develop the bar strength.

Additional Commentary

The following Commentary discusses selected sections of the 1994 UBC. Sections that do not reference a UBC section or paragraph present discussion of specific topics and do not refer to any specific UBC requirement.

C603 UBC §2105 Quality Assurance

Special Inspection is *always* required for masonry constructed using strength design provisions. Special inspection is optional for masonry constructed using working stress provisions if reduced allowable stress values are used. Special inspection should be used on as many structures as practicable, as the reduced allowable stresses may not adequately compensate for the probable lack of quality.

C604 General Design Requirements

C604.1 UBC §2106.1.12.4 Special Provisions for Seismic Zones 3 and 4

Item 1 Column Reinforcement. The nominal lateral reinforcement spacing at a maximum of 8 inches provides some confinement, but it is much less confinement than required for ductile reinforced concrete columns. This minimum confinement provides for a maximum usable strain of 0.006. If the maximum strain expected under real earthquake displacements (i.e., strains calculated using an R_w equal to 1) is greater than 0.006, greater confinement should be provided.

Item 2.1 Shear Walls: Reinforcement. Joint reinforcement shall not be used as the principal reinforcement to resist shears or lateral forces in lateral force resisting systems. The bond characteristics of the smooth wire reinforcement degrade rapidly under cyclic loading. There is a lack of experience in actual earthquakes on the performance of masonry with joint reinforcement.

Item 2.4 Stack Bond. Solid grouting and use of open end units is required to avoid the vertical lines of weakness that occur in stacked bond construction. The bond beam units are required to facilitate the flow of grout and to provide the assurance of obtaining a fully grouted construction.

C604.2 UBC §2106.2.14 Placement of Embedded Anchor Bolts

The performance of headed anchor bolts is generally superior to bent bar or plate anchor bolts. The test data for bent bar anchors loaded in tension or shear appears to have little consistency, whereas headed anchor bolts behave with a more consistent cone pullout-type failure similar to concrete, given that the headed anchor bolts have sufficient embedment, spacing, and end and edge distance.

C604.3 Working Stress Design and Strength Design Requirements for Reinforced Masonry

UBC §2106.3.2 Plain Bars. The use of plain bars in reinforced masonry construction is prohibited, except for joint reinforcement, because of the poor bond capacity, particularly under cyclic loading.

C605 Working Stress Design of Masonry

The working stress design method is an anachronism that has been patched together to give acceptable results for moderate levels of ground shaking. Use of the working stress design method for design of lateral force resisting systems should be minimized and the strength design provisions used where possible.

C605.1 ***UBC §2107.1.7 Shear Walls: Design Loads.*** UBC §2107.1.7 requires that the working stress seismic load be multiplied by a factor of 1.5. The 1.5 multiplier was introduced to provide an additional factor of safety based on observed performance of masonry walls in past earthquakes. Further experience with structures built to meet current detailing requirements and construction practices may permit reducing this factor. This method of accounting for uncertain behavior is taken from the masonry provisions of the 1994 UBC. Ideally, the material resistance uncertainty should be represented by a lower working stress or a smaller strength factor (e.g., the strength reduction factor $\phi = 0.6$ for reinforced masonry shear walls).

When overturning moments cause large flexural stresses, criteria similar to that used for reinforced concrete shear walls or masonry shear walls (UBC §2108) should be considered. Simply showing that the flexural strength capacity of a wall is greater than a factor of 1.4 times the overturning moment caused by code level forces is not sufficient to avoid the need for special boundary element confinement reinforcing. The inelastic deformation demands of a major earthquake may be significantly larger than the deformation computed at the factored moment value. Confinement similar to that necessary for a concrete wall is necessary to maintain the vertical load carrying capacity of the wall (see also Commentary Section C603.2).

C605.2 ***UBC §2107.2.2 Reinforcement.*** Working stress requirements are based on developing the reinforcement to anchor or transfer a force equal to the allowable stress times the nominal bar area. This is not consistent with the actual loading in seismic events. Since the actual stress in the reinforcing under the maximum seismic loading is unknown, the full strength of the reinforcement should be developed as is done for Strength Design. Use of the criteria in UBC §2108 strength design, accomplishes this requirement. All reinforcement shall be in accordance with the applicable provisions of UBC §2108 (e.g., reinforcement for shear walls shall be designed and detailed in accordance with UBC §§2108.1.4.6, 2108.2.2, and 2108.2.5.2).

C606 Strength Design of Masonry

C606.1 UBC §2108.1 General

The behavior of a masonry element, such as shear wall, when subjected to loads can best be described in terms of limit states. The masonry element can be idealized as evolving through identifiable states of behavior as loads are applied. This evolution can best be defined in terms of behavior and limit states. Table C606-1 identifies the behavior and limit states for a masonry element reinforced to behave in a ductile manner. A limit state exists at the end of each behavior state. For example, the first behavior state corresponds to the stress condition where the load-induced tensile stress is less than the modulus of rupture; hence, the wall cross-section is uncracked and the load-induced moment is less than the cracking moment capacity of the wall. The design criteria in this strength design section are a limit states criteria where the limit state is the strength limit state.

The ductile masonry element goes through several behavior and limit states prior to reaching its strength or ultimate limit state. For example, the load-induced moment, M, can be used as the variable to define the first limit state. The load-induced moment, M, will be equal to the cracking moment of the cross-section, M_{cr}. The second limit state is reached when M is equal to the yield moment, M_y, and the third limit state is reached when M is equal to the ultimate moment, M_u.

Each of these limit states can be of interest, depending on the design requirements. The first limit state corresponds to cracking of the cross-section, and thus to water penetration. This limit state can be viewed as a serviceability limit state. The second limit state defines the start of permanent steel deformation or structural damage. This limit state defines the structural damage or yield limit state. The third limit state defines the maximum usable masonry compressive strain, i.e., the point at which the load starts decreasing with increasing strain. This third limit state is a strength or ultimate limit state. The strength limit state is the state corresponding to the strength design provisions in the 1994 UBC.

C606.2 UBC §2108.1.2 Quality Assurance Provisions

Use of the strength provisions requires special inspection of all construction. The probability of satisfactory behavior of the constructed structural elements is improved by the required quality control provisions.

C606.3 UBC §2108.1.4 Design Strength

The axial load on a flexural member has a significant negative impact on ductility. For this reason, the strength reduction factor decreases with increasing axial load.

The strength reduction factor for shear for wall design for in-plane loads (shear walls) has two alternative values. The engineer should design, where possible, for the case where the nominal shear strength exceeds the shear corresponding to the development of the nominal flexural strength (ductile shear force). This case is controlled by a ductile flexural failure mode. The other case is where the nominal shear capacity is less than the ductile shear force, in which case the failure mode may be a brittle shear failure mechanism. The flexural mode of failure is preferred, hence, the higher capacity reduction factor.

C606.3.1 *UBC §2108.2 Reinforced Masonry*

UBC §2108.2.1.2 Design Assumptions. This UBC section sets the maximum usable masonry compression strain, ε_{mu}, as 0.003 for beams, piers, walls, columns, and wall frames. A higher limit may be used for wall frames *only* if confinement reinforcement is used. The shape of the uniaxial stress-strain curve for an unconfined masonry prism rises with increasing strain until it reaches a peak or maximum stress and then drops very rapidly with increasing strain, as shown in Figure C606-1. The strain corresponding to a maximum stress is called the peak strain; the larger strain corresponding to the point on the falling portion of the curve at which the stress is one-half of the maximum stress is called the maximum usable strain. The value of this maximum usable strength is set at 0.003 to be consistent with concrete design.

UBC §2108.2.2 Reinforcement Requirements and Details. Some research has suggested that the maximum bar size should be one less than the block thickness in order to minimize splitting, i.e., the maximum bar size for an 8-inch block wall is a #7 bar.

UBC §2108.2.2.6 Development. The UBC provisions for development and splicing are similar to the provisions proposed by ACI Committees 318 and 408 for the 1996 ACI Building Code [ACI-318], and are intended to develop the strength of the reinforcing. The failure mechanism is typically splitting of the masonry prism caused by radial tension stresses. Splitting is exacerbated by small cover and large bar size. There should not be a cap on the development length, as the larger reinforcing bars need very long development lengths to develop the bar strength.

C606.3.2 *UBC §2108.2.3 Design of Beams, Piers, and Columns*

UBC §2108.2.3.1 General. This section is intended to provide design criteria for reinforced masonry walls consisting of beams, piers, and columns. These structural elements describe the elements that make up a bearing or nonbearing reinforced masonry wall perforated by openings.

The probability of adequate behavior of structural assemblies constructed of these elements is improved by requiring quality control provisions. Masonry units are restricted to hollow units. Minimum prism strength is restricted to 1500 psi. Maximum strength that may be used in design computations is 4000 psi. Dimensional limits on beams, piers, and columns are specified. Minimum quantities and maximum spacing of reinforcement are specified. These arbitrary requirements were included to ensure that the constructed masonry reasonably conforms with the construction of the tested specimens.

UBC §2108.2.3.2 Design Assumptions. The member forces shall be based on an analysis that considers the relative stiffness of all structural members. An analysis similar to that made for a frame composed of the beam, pier and column elements would conform to this requirement. The joint elements, the material common to the beam and column or pier, could be considered as a rigid element. The effect of cracking on member stiffness must be included in the analysis. This requirement requires an iterative process. The estimate of effective moment of inertia, I_e, is used to calculate the moments, M_s, in the member. The cracking moment, M_{cr}, is calculated as the product of the section modulus of the uncracked section and the modulus of rupture. The modulus of rupture of grouted hollow unit masonry is assumed to be 235 psi. This value was derived from experimental testing. If the initial estimates of effective moment of inertia are significantly changed by the results of the preliminary analysis, the revised estimates of effective moment of inertia are used in the subsequent analysis until closure of the iterative procedure is obtained.

The interstory drift ratio calculated by the analysis of the system shall be compared with the drift limits specified in the general seismic design sections.

Strength computations for beams, piers, and columns are based on an assumption that strain in the masonry and reinforcement in compression, and in the reinforcement in tension, is proportional to the distance from the neutral axis of the flexural section. The maximum strain used in calculations at the extreme compression fiber shall not exceed 0.003. The strain of the extreme tension reinforcement may exceed yield strain, but the stress in reinforcement shall be taken as the specified yield stress. The strength of the element shall be calculated by applicable conditions of equilibrium. The tensile forces in the reinforcement and the appropriately load factored axial loads shall be in equilibrium with the compressive forces in the assumed compression block. The compression block may be assumed to be rectangular and having a uniform stress of 0.85 of the specified compressive prism strength. The assumed size of the rectangular compression block is limited to 0.85 of the distance from the neutral axis to the extreme compressive fiber multiplied by the width of the element. The computation of flexural strength shall neglect the tensile strength of the masonry. Deformation calculations may include the tension capacity of masonry by use of a tension stiffening model. The equation for computation of the effective moment of inertia considers tension stiffening effects in cracked zones and the effects of uncracked zones in the length of the element.

UBC §2108.2.3.3 Balanced Reinforcement Ratio for Compressive Limit States. This section specifies a procedure for limiting the quantity of reinforcement that can be placed in a member subjected to flexural or flexural and axial loads. Specification of a limit on the combination of axial loads and quantity of reinforcement allows nonlinear curvature of the yielding cross-section. An experimental specimen subjected to increasing curvature in the yielding zone would eventually result in increasing the compressive strain at the extreme fiber to a strain that exceeds the usable strain. Two restrictions are used in conjunction to minimize the probability of exceeding the usable compressive strain in the element: 1) a limitation on the quantity of reinforcement; and 2) a drift limit. Lightly reinforced elements with small axial loads will be restricted in curvature by the drift limit. Elements with large axial loads will have severe restrictions on the allowable quantity of reinforcement. The second restriction can be met by increasing the size of the masonry cross-section.

The method specified for calculation of a balanced reinforcement ratio for compressive limit state is an empirical method selected for simplicity of calculations. The axial load used for the computation has a load factor of 1.0 for dead and live loading. The axial loading caused by earthquake or wind is factored to account for the probable overstrength of the system. The balanced reinforcement ratio is calculated as a uniformly distributed quantity of reinforcement. Use of uniformly distributed reinforcement is required by this design section. The contribution of compression reinforcement is not considered. If the compression reinforcement were considered, the empirical method would not have a rational solution. However, the contribution of compression reinforcement may be included in strength computations.

UBC §2108.2.3.6.2 Nominal Shear Strength. The shear strength is calculated as the sum of the shear strength of the masonry cross-section and the strength provided by the transverse shear reinforcement. The value of the shear strength of the masonry shall be assumed to be zero when the full cross-section is in tension for factored loading. The shear strength of the masonry is limited to 25 psi when the required strength, M_u, exceeds 70 percent of the nominal moment strength. These requirements are intended to limit the allowable shear strength contributed by masonry when the flexural strength provided and the required flexural strength are nearly equal. The calculation of required moment is based on an R_w of 3 to better approximate the probable moment caused by seismic loading.

UBC §2108.2.3.7 Reinforcement. The maximum spacing of transverse reinforcement, when required, is limited to one-half of the depth of the member. This requirement is specified to decrease the probability of a shear mode of failure.

The flexural reinforcement must be distributed uniformly across the cross-section, except as specified for beams, and must be a symmetric pattern if the section is subject to load reversals. The usable strength at any section is limited to one quarter of the maximum moment strength provided. The maximum quantity of reinforcement is limited to 50 percent of that calculated for the compressive limit state. These restrictions are intended to improve the capacity of the system to tolerate nonlinear curvature in probable yield zones.

UBC §2108.2.3.8 Seismic Design Provisions. The required lateral load resistance in any line or story level shall be provided by shear walls or moment frames when strength design methods are used. Cantilever shear walls and wall frames are known to have desirable nonlinear behavior. The beams, columns, and piers in line with shear walls and/or wall frames are designed for the loading induced by their relative stiffness. The forces in the beams, columns, and piers are calculated by the relative stiffness analysis. The shear walls or wall frames are also required to provide 80 percent of the combined stiffness of any line or story. This requirement generally would require that a shear wall be placed in line with the masonry wall composed of beams, piers, and columns. This requirement should not be construed to mean that the assemblage of beams, piers, and columns need not be analyzed. Analysis will determine the stresses induced in beams, columns, and piers by drift of the lateral load-resisting system.

An exception to these seismic design provisions is made for structures whose behavior under seismic loads is essentially elastic. The assemblage of beams, columns, and piers may be used for resistance to seismic loading if the R_w used for determination of seismic loading does not exceed 3. A further limitation on this usage is that the joints, the intersections of beams and columns or piers, shall conform to all of the joint provisions specified for wall frames. Systems comprised of beams, piers, and columns that are not restricted to development of yielding in beams have a greater probability of having a shear mode of failure. The reduction in the maximum value of R_w is intended to provide an increase in the capacity of the system.

C606.3.3 ***UBC §2108.2.4 Wall Design for Out-of-Plane Loads.*** Slender wall renamed.

C606.3.4 ***UBC §2108.2.5 Wall Design for In-Plane Loads.*** The new name for shear walls.

C606.3.5 *UBC §2108.2.6 Wall Frames*

UBC §2108.2.6.1 General Requirements. The masonry wall frame is a new lateral force resisting system first approved for the 1993 UBC Supplement. The wall frame provides the engineer with a lateral force resisting system that can accommodate window or storefront openings where solid shear walls are undesirable. The design criteria requirements of this section were established to require all flexural yielding and inelastic energy dissipation to occur in the beams; no yielding is to occur in the piers.

Because the wall frame is a new system, there are numerous material and geometric restrictions. The geometric constraints placed on beam dimensions are such that most of the deformations are flexural deformations rather than shear deformations. Also, piers, which are the counterpart of a column in a concrete frame, have limitations on the allowable axial load. These geometric and material limits were set to keep the wall frames within the boundaries of the results of available research.

UBC §2108.2.6.2.5 Flexural Members (Beams). Longitudinal reinforcing steel is required to be distributed evenly over the depth of the beam rather than concentrated at the top and bottom, as is common for reinforced concrete beams. This even distribution governs the behavior of the beam. Instead of an essentially elasto-plastic moment curvature relationship where essentially all the tension steel yields simultaneously, the transition from first reinforcement yield to ultimate strain is more gradual as successive reinforcing bars yield. The resulting moment curvature relationship is a gently rising curve from first yield to ultimate.

The minimum reinforcing ratio required for the beams ensures that the ultimate moment capacity will be greater than the cracking moment. The ratio of ultimate moment to cracking moment was set at 1.4 to account for uncertainty in cracking moment. The maximum reinforcing steel ratio allowed ensures that the flexural steel will yield well before the masonry reaches ε_{mu} in order to ensure ductile behavior.

Note that the beams lack confinement reinforcing. Confinement reinforcing for masonry in compression only becomes effective when the compressive strain in the material become large and results in large lateral Poisson strains. The ductility of a structural member is a function of: 1) quantity of tension reinforcing; 2) axial compressive load; and 3) maximum usable compressive strain. The beams in a wall frame have little or no axial load and the quantity of tension reinforcing is limited. Confinement reinforcing would increase the maximum usable masonry compressive strain, but does not appear to be required for the configurations tested. The lack of confinement reinforcing is an additional reason for the geometric and material limits.

UBC §2108.2.6.2.6 Members Subjected to Axial Force and Flexural (Piers). The piers are basically slender shear walls. Hence, many of the requirements are similar to those for shear walls. Confinement must be provided if the compressive strains at factored forces determined using a R_w of 1.5 exceed 0.0015. This procedure is intended to determine real strains at realistic earthquake loads, and provide confinement as necessary. Confined masonry has a much larger usable compressive strain than unconfined masonry. See Figure C606-1.

UBC §2108.2.6.2.7 Pier Design Forces. The pier nominal moment strength must exceed the pier moment corresponding to the development of beam plastic hinges. This criteria preserves the strong column-weak beam assumption inherent throughout these Requirements. The factor of 1.6 was considered necessary to ensure this relationship because of limited test data. The piers are intended to remain essentially elastic except at the base.

UBC §2108.2.6.2.8 Shear Design. Shear design includes both a masonry and reinforcing steel contribution similar to shear walls. The nominal shear strength is required to be greater than 1.4 times the shears corresponding to beam flexural yielding in order to preclude a brittle shear failure mode.

UBC §2108.2.6.2.9 Joints. There are strict geometric limits for the joint. The reasons are: 1) the joint must provide sufficient load-carrying capacity to transmit the forces from the beams to the columns; 2) the stresses in the joint must be low enough to allow the beam longitudinal steel to be developed within the joint; 3) the shear forces in the joint must not exceed the shear capacity; and 4) there is limited test data. Since there is no confinement reinforcement per sė in the joint, the development lengths for cyclic behavior of reinforcing steel in grout dominate the geometry. The beam longitudinal reinforcing must be adequately developed in the joint to prevent slip and joint degradation.

Reference

Brandow, Gregg E., Gary C. Hart, and Ajit Virdee, *Design of Reinforced Masonry Structures*, Concrete Masonry Association of California and Nevada, Citrus Heights, California, 1995.

Table C606-1. *Behavior and Limit States—Ductile Material*

State	Description
Behavior State 1	Uncracked cross-section and $M < M_{cr}$
Limit State 1	$M = M_{cr}$ and the maximum stress in the masonry is equal to the modulus of rupture
Behavior State 2	Cracked cross-section with strain in the steel $\varepsilon_s < \varepsilon_y$ and $M_{cr} < M < M_y$
Limit State 2	$M = M_y$ and strain in the steel reinforcing farthest from the neutral axis $\varepsilon_s = \varepsilon_y$
Behavior State 3	Cracked cross-section with strain in the steel reinforcing farthest from the neutral axis $\varepsilon_s > \varepsilon_y$, but the maximum strain in the masonry $\varepsilon_m < \varepsilon_{mu}$ and $M_y < M < M_u$
Limit State 3	$M = M_u$ and the maximum strain in the masonry $\varepsilon_m = \varepsilon_{mu}$

where: M = applied moment

M_c = cracking moment capacity of cross-section

M_y = yield moment capacity of cross-section

ε_s = steel strain

ε_m = masonry strain

ε_{mu} = maximum usable masonry strain

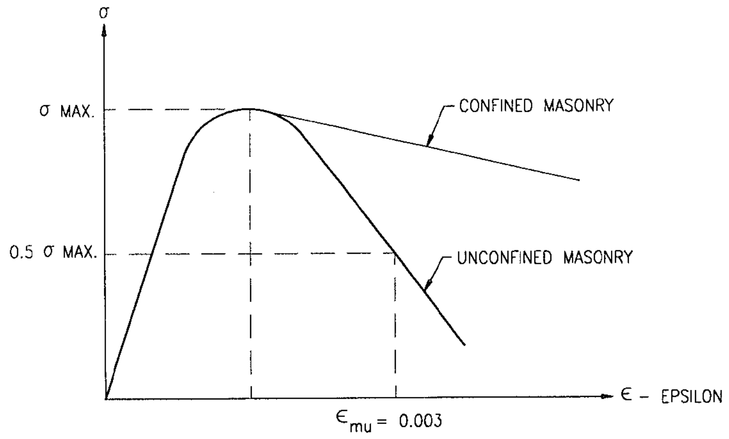


Figure C606-1. *Compressive stress strain diagram for masonry*

CHAPTER 7

Structural Steel

C701 UBC §2211.1 General Requirements

SEAOC endorses the code requirements of the 1994 Uniform Building Code (UBC), with the exception of the Modifications recommended in Section 702 (not yet adopted into the UBC). This sixth edition of the Blue Book also offers general commentary on UBC provisions relevant to steel construction.

Neither the American Institute of Steel Construction (AISC) "Specification for Structural Steel Buildings, Allowable Stress Design and Plastic Design," (ASD) dated June 1, 1989, nor "Load and Resistance Factor Design (LRFD) Specification for Structural Steel Buildings," dated December 1, 1994 [AISC, 1994] encompasses detailed design of steel buildings or structures subjected to substantial earthquake effects. During a major seismic event, reliance is placed upon the building's performance at deflections significantly beyond those at levels calculated using code specified force levels. The AISC specifications are formulated on the basis of performance primarily in the elastic range, and do not offer provisions that would promote stability of steel structures deflected well into the inelastic range.

These Requirements are written for working stress design evaluation. AISC has published seismic provisions for LRFD. This document, entitled "Seismic Provisions for Structural Steel Buildings," dated June 15, 1992 [AISC, 1992], is roughly parallel with many of the UBC requirements, and contains an excellent commentary. The AISC seismic provisions may be adopted (in part or full) in a future edition of this Blue Book when Blue Book Requirements become strength based.

This Chapter includes Requirements for all of the more common structural steel systems; including ordinary and special moment resisting frames, concentrically braced frames, the newly developed special concentrically braced frames, eccentrically braced frames, and stud wall systems.

In many UBC provisions, adequate test data does not exist to formulate precise limits. In those cases, these recommendations are based on experience and engineering judgment. An excellent overview of the critical issues concerning connections and bracing design criteria is given by Popov [Popov, 1988]. The many issues regarding moment frame design (both special and ordinary) that were brought to light by the performance of steel frame buildings in the 1994 Northridge earthquake are addressed in the FEMA Interim Guidelines [FEMA, 1995].

C701.1 UBC §2211.2 Definitions

The definitions presented in this section are for the purpose of this chapter only, and where a term is not identified, the definitions in the other sections of these Requirements and in the referenced standards are to prevail, in that order.

Allowable Stresses. With respect to allowable stresses, the AISC, ASD A5.2 applies such that the one-third increase in allowable values may be used for load combinations that include seismic load.

Gravity Loads. When gravity loads are prescribed in this chapter, they are as defined in the UBC: Dead load plus live load including any reductions for tributary area, and snow load as per UBC §1603.6.

C702 Recommended Modifications to 1994 UBC

The extensive damage to steel moment resisting frames in the 1994 Northridge earthquake resulted in an emergency code change that suspended the prequalified status of the standard moment connection detail. Detailed discussion of this change is included in Commentary Section C706.

702.1 Prequalified Moment Connections

Delete: ***UBC §2211.7.1.2*** The extensive damage to steel moment frame connections in the 1994 Northridge earthquake resulted in an emergency code change to the UBC. This emergency change suspended the prequalified status of the standard moment connection detail. Because of this, UBC §2211.7.1.2 was deleted. See Commentary Section C706 for further discussion.

702.2 Connection Strength

Modify: ***UBC §2211.7.1.3 Connection Strength.*** Steel moment frame connections must meet the general performance requirements for strength and inelastic rotation of UBC §2211.7.1.1. This must be demonstrated by cyclic tests or calculations since the previously prequalified status is no longer allowed. See Commentary Section C706 for further discussion.

Additional Commentary

The following Commentary discusses selected sections of the 1994 UBC. Sections that do not reference a UBC section or paragraph present discussion of specific topics and do not refer to any specific UBC requirement.

C703 Steel Properties

C703.1 UBC §2211.4.1 Materials

Currently, the structural steels permitted for use in seismic force resisting frames and structures include the following: A36, A441, A500, A501, A572 (grades 42 and 50), and A588. This list has been compiled by considering the inelastic properties and weldability of currently available steels. These structural steels generally have the following characteristics:

1. Ratio of tensile strength to yield strength in the range of 1.2 to 1.8.
2. Pronounced yield stress-strain plateau.
3. High inelastic region strain capability.
4. Tension elongation in a 2-inch gage length of greater than 20 percent.
5. Favorable weldability for ductile behavior.

Steels not listed above, including steels of greater than 50 ksi specified yield strength, should not be used until further research indicates that their inelastic properties will result in adequate ductile behavior.

Adequate ductile capacity is not the only property important to good seismic behavior of steel frames. It is also important that the yield strength of materials incorporated into the structure be controlled within tolerable and/or anticipated limits, so that unintentional overstrengths do not result in inelastic behavior other than that intended in the design.

As an example, special moment resisting frames designed for inelastic behavior to be controlled by flexural yielding of beams, as opposed to columns, could experience exactly the opposite behavior if the yield strength of the beams is inadvertently larger than that used in the calculations. Changes in mill technology have resulted in a steady increase in the expected strength of A36 steel, with most mills now commonly producing steels to this specification that have yield strengths in excess of 50 ksi. Since the steel used on a project may be produced by a variety of mills, wide variation in yield strength can occur throughout a building. Note that variation in yield strength also occurs for steels produced by the same mill. The designer is cautioned that the following problems may occur from the incorporation of significantly overstrength material into lateral force resisting systems:

1. Members that are marginally compact at the specified yield strength may not behave in a compact manner at the actual yield strength. As a result, local buckling and crippling of slender elements of cross-sections could occur, resulting in undesirable inelastic behavior and substantial reduction in energy dissipation capacity.
2. Lateral bracing provided for compression flanges based on specified yield strength may not be adequate for the actual yield strength, resulting in potential for inelastic behavior of frames dominated by flexural-torsional buckling.

3. Connections designed for member-specified yield strength may be inadequate for the actual member strength (for example, consider the design procedure for shear tab connections in the AISC manuals).
4. Columns proportioned with marginally adequate axial strength to resist the full anticipated plastic load on a frame may be inadequately sized for the actual yield capacity of the frame.
5. Eccentric braced frames, which are intended to be controlled by yielding of link beams, may actually be controlled by buckling or yielding of attached braces and/or columns.

In August 1992, representatives of the steel industry met with members of SEAOC and SEAW (Structural Engineers Association of Washington) to discuss these issues. This meeting led to a research investigation to collect statistical data on A36, A572 GR. 50, and so called "dual certification steels that simultaneously meet the requirements of both A36 and A572 GR. 50. The resulting Structural Shape Producers Council (SSPC) report [SSPC, 1994] was based on information gathered from over 57,000 mill certifications. A summary of statistics is reprinted from this report as Table C703-1. The mean yield stress for A36 material is just below 50 ksi, while for A572, this value is 57.6 ksi. Note that the yield stress varies somewhat between the various group designations, with generally lower values for the larger sections.

The effect of these results on the design of lateral force resisting systems must be properly considered. Some possible methods to address this issue include the following:

1. Limiting the use of A36 material in lateral force resisting systems, especially in combination with elements of other yield stresses.
2. Assuming overstrength material is present when calculating requirements for section compactness and lateral bracing, and when computing maximum demands on connections and adjacent elements.
3. Specifying maximum yield strengths for material incorporated into a project. It should be noted that the industry does not currently control the maximum yield strength of material, and this added requirement may result in a significant cost premium. Steel producers have resisted any limitations on the maximum steel yield strengths.

C703.2 UBC §2211.4.2 Member Strength

Whenever the element "strength" is referred to in the provisions of UBC §2211, the basis is as defined in UBC §2211.4.2. This is not the strength as defined in static maximum elastic strength design, or the maximum plastic strength as defined in limit design. This is simply the strength referred to in specific UBC provisions in this UBC §2211.4.2.

The strength specified is used as a limit when dealing with the strength and stability of the element itself. In subsequent sections, material overstrength and strain hardening effects are accounted for by use of a multiplier on the member strength. This is done to determine the maximum load that the element in question can transfer to an adjacent element.

C704 Column Requirements

The provisions of UBC §2211.5 apply to columns in all structural systems. They are gathered here for ease of presentation and to emphasize the importance of columns to the seismic resistance of a building or structure. The integrity of a structural system in postelastic seismic excursions is strongly dependent on the axial capacity of its columns.

C704.1 UBC §2211.5.1 Column Strength

During the maximum expected earthquake the column axial forces, calculated in accordance with the provisions of Chapter 1 of these Requirements, may be significantly exceeded. To provide added capacity in the axial strength of columns, a multiplier of $3(R_w/8)$ is prescribed for the seismic axial loads in the two additional load combinations. The multiplier is primarily intended to provide for the effects of overturning from forces that may exceed those calculated from the basic seismic provisions.

The two load combinations, the first for compression with reduced live load, and the second for tension with reduced dead load and no live load, are to be resisted by the axial strength of the column and the effects of flexural loads need not be included.

The column axial strength formulas are given in UBC §2211.4.2. The column splices and base plate hold-down capacity should have at least the strength to resist the tension force resulting from the second load combination.

Exceptions:

1. For this stability check, the axial forces need not exceed the maximum axial force that can be delivered to the column by the structural system when stressed to its strain hardened strength (defined as 1.25 times the strength of the elements of the structural system).
2. The axial overturning force need not exceed the uplift strength the foundation system is capable of developing.
3. This check need not be made for moment resisting frames when the ability of the beams to transmit axial force to the column is limited by plastic hinges forming first in the beam. This is addressed by allowing the exception if the "strong column-weak beam" relationship (UBC §2211.3.3) is satisfied, and if, at the same time, the column axial stress f_a due to the other applicable load combinations is less than $0.3F_y$.

When utilizing exception 2 for the case of columns supported by pile foundations, it should be recognized that the actual tension or uplift resistance of the piles may be much more than the value employed for use in the design of the foundation.

C704.2 UBC §2211.5.2 Column Splices

Column splices, as a minimum requirement, must be able to transmit the prescribed axial forces in addition to the shears and moments occurring at the splice location.

In order to prevent fracture, more stringent provisions are required for splices that are required to transmit net tension forces. The Requirements of this section also require partial penetration welds that are subjected to net tension under Section C704.1.2 to be designed for forces in excess of the design forces and that the splice location be located at least 3 feet from point of maximum moment. The engineer should recognize that these splice Requirements are minimum requirements intended for common frame configurations. Extra strength and detailing may be required for conditions such as: columns in tall stories, large changes in column sizes at the splice, or where there is the possibility of a single curvature moment gradient in the column. Analyses have shown that the typically assumed condition of inflection points near the mid-height of each story of the columns may not be appropriate during response to lateral forces [Paulay, 1983]. Under such conditions single curvature moment gradients may occur over multiple stories, resulting in large moments at locations that would typically be expected to be near a point of inflection.

C704.3 UBC §2211.5.3 Column Slenderness Evaluation

Formula 1.6-1a of the AISC Specification requires that the allowable axial stress, F_a, be determined from consideration of the slenderness ratio, KL/r.

For braced frames, or frames where joint translation is restrained by shear walls or other structures having adequate lateral stability, the minimum effective length factor, K, for compression members should be taken as unity, unless analysis demonstrates that a smaller value may be used; this analysis should consider structural deformations due to maximum expected ground motion.

For unbraced frames where the lateral stability is dependent on the bending stiffness of rigidly connected beams and columns, the AISC Specification requires determination of the effective length by prescribed procedures. The procedures are based on the restraint provided at each end of the column with the limitation that the effective length may not be less than the actual unbraced length.

Note that the AISC Specification assumes no p-delta effects occur for braced frames or shear walls, even though shear drift for these systems is not necessarily small and chord drift may be large. The AISC Specification assumes no P-delta control for moment frames even though moment frames are required to be designed with the same drift and P-delta controls as required for other systems. These Recommendations consider drift in seismic design and require consideration of P-delta effects. Therefore, the complications of designing for unknown side sway effects are not needed, and using an effective length factor “K” of 1 provides some simplicity in the complicated design process.

C705 UBC §2211.6 Ordinary Moment Frame Requirements

Ordinary moment resisting frames (OMRFs) of structural steel are moment resisting space frames that do not meet the special detailing requirements for ductile behavior contained in Section C706. Although the design lateral loads for OMRFs are twice those of special moment resisting frames (SMRFs), based on the observed Northridge earthquake damage, this is not sufficient to recommend their use.

Based on observations of Northridge earthquake damage and some analysis, the consensus of the SAC Phase I Investigation participants was that the SMRF prequalified connection appears to have often failed at or before development of the nominal plastic capacity of the girder [SAC, 1994]. Research has also shown that there is no consistent linkage between the R factor and rotation demands. Further, because of the drift limitations, the design of welded steel moment frames in mid- and highrise buildings is governed by stiffness rather than strength.

Therefore, the use of an R_w of 6 versus 12 may not make a significant difference in design. This may not allow the connections to remain elastic, as the imposed ductility demands on the frame connections are quite sensitive to the site ground motion intensity and duration, as well as system redundancy and overstrength. Nonetheless, occasions do arise when it is not possible or not essential to meet all of the detailing requirements of Section C706. Examples include light metal buildings, usually limited to one or two stories, and open industrial structures. Therefore, SEAOC decided that ordinary moment frames should continue to be permitted, but only in limited applications.

Although it is not required that the Requirements of Section C706 (except C706.1) be met for an ordinary moment frame, it is SEAOC's recommendation that the detailing Requirements of Section C706 be met, at least to the extent that practicality and good engineering judgment would suggest. It is particularly suggested that those Requirements that deal with element stability, such as C706.3, 706.7, and C706.8, should at least be considered. In light of the Northridge damage, consideration should also be given to improving the ductility of the moment connections themselves.

C706 UBC §2211.7 Special Moment Resisting Frame (SMRF) Requirements

Prior to the January 17, 1994 Northridge earthquake, the special moment resisting frame (SMRF) was believed to be a proven, reliably ductile structural system. SMRF buildings constructed during the previous two decades typically utilized beam-to-column connections in which beam flanges were attached to the columns with complete joint penetration welds, and beam webs were attached using high-strength bolts. During the Northridge earthquake, more than 100 buildings with welded steel moment resisting frames suffered beam-to-column connection failures. Observations of these buildings indicated that, in many cases, fractures initiated within the connections at very limited levels of inelastic behavior.

Immediately following the Northridge earthquake, tests were performed at the University of Texas on a series of beam-column connections fabricated using heavy column and deep beam sections typical of some modern construction. Precautions were taken to ensure welds were of good quality, and contrary to common practice, backing bars were removed from the bottom flanges. Despite these precautions, the test specimens failed at very low levels of inelastic loading. Review of literature on historic tests of these connection types indicated a significant failure rate in past tests as well, although these had often been ascribed to poor quality in the specimen fabrication. Based on the Northridge data, the subsequent testing, and a review of historic testing, it was concluded that the prescribed connection in UBC §2211.7.1.2 was fundamentally flawed and should not be used for new construction in areas of high or moderate seismicity, pending the results of research to define the conditions under which it can perform in a reliable manner. Consequently, in September 1994, the International Conference of Building Officials (ICBO) adopted an emergency code change to the 1994 edition of the UBC governing girder-to-column connections. This code change is discussed in detail in Section C706.1.3, below

The basic philosophy of the design of SMRFs is simply that nonductile failure modes should be prevented, and that inelastic deformations should be concentrated in regions that have a capacity for ductile behavior. Thus, column buckling and connection failures are to be avoided. At the same time frame stiffness, and therefore frame stability, must be maintained. These requirements are intended to minimize the possibility of premature failure of nonductile elements.

Ductile behavior and inelastic energy absorption may be allowed to occur in three elements of a moment frame. These are: the beams and the columns in which flexural hinges may form; and the joint panel zone, in which shear yielding may take place. Within limits, inelastic behavior of any or a combination of these three elements is permitted, with the final choice being left to the engineer. Depending on which of the three elements is chosen to have the lowest inelastic threshold, different constraints must be observed and different problems present themselves to the engineer.

Although considered by many engineers to be the least desirable choice, yielding in columns is permitted, although subject to the limitations of Section C706.5. If this option is selected, then the girder-column joint bracing requirements of Section C706.7 must be observed.

If the engineer elects to permit yielding of the panel zone as the first inelastic mode, then the limitations of Section C706.2.1 must be observed and the resulting increase in frame flexibility must be accounted for in the drift calculations, as required in Section C706.10. If the designer chooses to avoid either column yielding or the greater flexibility implied in a yielding panel zone for reasons of overall frame stiffness, then the yield hinge behavior is transferred to the girder. In that case, the girder-column connection and beam stability become the elements of greatest concern.

Section C706 provides special attention to the three possible yielding and energy-absorption elements through a series of special requirements designed to promote strength and stability of yielding elements during inelastic seismic excursions. In a dual framing system, the SMRF is designed to carry its share of lateral forces, as determined by stiffness, but not less than 25 percent of total shear. In many cases back-up frames may be stressed at relatively low levels and yielding of frame panel zones is unlikely to contribute significantly to drift. It is therefore likely that panel zone doublers would not be required and the strength of the panel zone may become the determining joint strength for the purposes of detailing.

C706.1 Girder-to-Column Connection

The intent of this section is to ensure that in the performance of the overall seismic force resisting system, the yield hinge occurs in a location other than in the connection itself.

C706.1.1 ***UBC §2211.7.1.1 Required Strength.*** This section requires that girder-to-column connections develop the lesser of the girder strength or the moment corresponding to the panel zone shear strength, the latter as determined by UBC equation 2211-1. It is recommended that the connection develop the strain-hardened panel zone (see Commentary Section C706.2.1). The option of developing the flexural strength of a yielding column was rejected by SEAOC. It was believed that permitting this latter option would offer excessive encouragement to a practice (strong girder-weak column) that SEAOC wished to limit, or at least to discourage.

> ***Exception***: An exception is provided for connections that are not designed to contribute flexural resistance. When, in the determination of frame strength, connections are designated as non-moment resisting, the requirement is that their inelastic rotation capacity be adequate to satisfy the requirements of UBC §1631.2.4. If non-moment resisting connections such as shear tabs are used, where the tab is welded to the column and bolted to the beam web, care should be exercised to detail the tab thickness and weld such that the tab will yield due to the actual flexural action caused by the bolted web connection. Induced stresses should not cause fracture of the weld to the column.

C706.1.2 ***UBC §2211.7.1.2 Connection Strength.*** The 1988 edition of the UBC introduced prescriptive design provisions for SMRF connections. In response to concerns about their performance during the 1994 Northridge earthquake, the Board of Directors of ICBO passed an emergency code change that deleted provisions prescribing the requirements of beam-column joint connections for use in Seismic Zones 3 and 4. In its place, Requirements were added to require demonstration of connection adequacy by testing or calculation. The uncertainty regarding the nature of the causes and solutions to the connection problem suggested the use of broad, general language in the emergency code change, allowing latitude in interpretation by structural engineers and building officials.

A research program was developed under the auspices of SEAOC, ATC, and CUREe, otherwise known as the SAC Joint Venture. The SAC Joint Venture was commissioned by the Federal Emergency Management Agency (FEMA) and the California Office of Emergency Services (OES) to investigate the damage to steel moment resisting frames caused by the Northridge earthquake, to disseminate relevant information to practicing engineers, and to develop Interim Guidelines for the inspection, evaluation, and repair of buildings affected by the Northridge earthquake and for the design of new buildings. Those Interim Guidelines were published by FEMA in September 1995. This Chapter 7 Commentary specifically addresses the measures needed to comply with the 1994 Emergency Code Change. Other chapters of the FEMA Interim Guidelines contain information critical to the proper design, fabrication, and erection of steel moment resisting frames.

The procedures and recommendations presented in the FEMA Interim Guidelines have not been condensed for reproduction into this document. The information and recommendations provided in the SAC document should be used in conjunction with the remainder of the Commentary in this Chapter. It should be recognized that there are design issues where the FEMA Interim Guidelines and this Commentary overlap, but may not be completely consistent, since the two documents were being developed simultaneously. The engineer is strongly urged to obtain the Interim Guidelines document for use in the design of any steel moment resisting frame building. Interested parties may obtain material published by SAC by contacting:

Structural Engineers Association of California
555 University Avenue, Suite 126
Sacramento, California 95825
Telephone (916) 427-3647 Fax (916) 568-0677

It is unlikely that universal prescriptive design standards will produce acceptable seismic frame connections. Those that use the Interim Guidelines should recognize that there is considerable professional debate regarding the specific steps needed to achieve acceptable seismic performance in seismic frame beam-column connections. Therefore, it is recommended that these Interim Guidelines be applied with sound professional judgment to reflect the specific conditions present in the subject project and not adopted as unvarying design requirements. In addition, engineers must ensure that the design decisions they make in an attempt to avoid the connection damage observed following the Northridge earthquake do not develop connections that possess excessive strength at the expense of an acceptable level of ductility. Achieving this balance will represent a significant challenge.

C706.2 Column-Girder Joint Panel Zone

C706.2.1 ***UBC §2211.7.2.1 Panel Zone Strength.*** Since 1969 the UBC has required that beam-to-column connections develop the full plastic capacity of the beam, but this requirement was specifically not extended to the joint panel zone. This was based on panel zone tests [Becker, 1971; Slutter, 1981], which showed their ample ductility. However, previous SEAOC Commentary implied that the panel zone should also, as a minimum, develop the beam, rather than the prescribed seismic forces. This section is intended to clarify this issue.

During recent years, many cyclic tests have shown the ductility of shear yielding in panel zones through many cycles of distortion beyond the demands of code design forces. Since the panel zone need not support beam hinging to make a moment frame ductile, the question remains: what force should the panel zone be required to support? The UBC has required the panel zone to support code forces at a shear force of 1.33 x 0.40 $F_y d_c$, or 0.53 $F_y d_c$. This seems inconsistent with a yield strength capacity assumed as 0.55 $F_y d_c$. However, many panel zone and beam tests have shown a strain-hardened shear capacity well over 1.0 $F_y d_c$, depending on the strain. This apparent contradiction is the result of assuming the shear area as only d_c, whereas the flange area significantly contributes to beam shear capacity, particularly in the inelastic range.

It was decided to use UBC eqn. 2211-1 [Krawinkler, 1978], which is for the condition of a fully yielded panel zone and includes the contribution of the column flanges, to indicate shear capacity at significant yielding of the panel zone. A 1.85 factor was agreed upon as a compromise (an average of 1.70 and 2.00) to be applied to the prescribed panel zone force to establish the relationship between the first shear yield and prescribed seismic forces. However, a UBC §2211.7.2.1 exception provides that the panel zone need not develop more than 0.8 times the sum of the beam plastic moment capacities at the joint. The 0.8 factor is introduced to reflect the participation of gravity loads [Popov, 1987]. While these criteria may represent the SEAOC position, it is important to recognize that there are important alternative concerns relative to panel zone resistance [Popov, 1987].

Section C706.10 permits a simplified drift calculation if the panel zone strength can develop more than $0.8\Sigma M_s$. The panel zone shear strength also need not exceed $0.8\Sigma M_s$. This level of panel zone strength will automatically satisfy the Requirements of both this section and section C706.10 without extra computation.

Lesser panel zone strengths are permitted, but when used, it is required that the additional loading and panel zone stress and their extra contribution to drift (and thus to P-delta effects) be accounted for. A simple method for the calculation of drift due to panel zone shear is provided in [Krawinkler, 1978]. Also, significant panel zone yielding can create large "kink" rotations at the girder-to-column flange connection and thus lead to premature tearing of the flange weld due to high stress concentration [Popov et al., 1986].

Again, based on Krawinkler [1978], when two beams of different depth frame into the column in interior joints, it is conservative to use the larger value of d_b in UBC equation 2211-1.

It should be noted that the 1.85 factor was selected based on the pre-Northridge earthquake welded flange/bolted web connection. This factor should be used with caution for reinforced connections that transfer higher shear forces into the panel zone. Moderate panel zone yielding may have a beneficial effect on SMRF performance, as it can relieve some of the plastic rotation demand on the beam. A reasonable design goal would be to achieve a condition that induces panel zone participation in the inelastic response. The panel zone should not be so weak that excessive yielding occurs, because of the concern for local "kinking" of the column flanges adjacent to the beam flange weld and the possible development of a story mechanism. However, very stiff panel zones and thick column flanges minimize the effective participation of this region in providing ductility and increase beam connection ductility requirements. Further, stiff panel zones may result in higher triaxial residual stresses in the welds.

UBC equation 2211-1 was intended to induce a panel zone distortion on the order of four times the yield level. However, there is very little evidence of any significant panel zone yielding having occurred in buildings inspected following the Northridge earthquake. Additional research on the adequacy of the current limitations on panel zone strength, especially for reinforced joint configurations, is required to ensure that the stiffness and strength of the panel zone is adequately controlled.

C706.2.2 UBC §2211.7.2.2 Panel Zone Thickness. UBC equation 2211-1 is based on tests [Krawinkler, 1978] that indicate that if the ratio of the sum of the panel zone depth plus width to the thickness is less than 90, buckling will not reduce the shear capacity below $F_y\, dt$ even under repeated cyclic loading.

C706.2.3 UBC §2211.7.2.3 Panel Zone Doubler Plates. Selecting a heavier column may be more economical than adding doubler plates. Nevertheless, it is desirable to establish a standard method of installing doubler plates in order to provide code control and assist uniform testing. Fillet welding should be encouraged, since it involves less built-in weld stresses. The cost of connecting doubler plates varies greatly with the welding used, and fillet welds are much more economical than butt welds if the fillet welds are not large. Fillet weld tests [Butler and Kulak, 1971] and doubler fillet welds have shown capacities much larger than those allowed by the UBC, and fillet welds stressed parallel to their length have a large ductility. Therefore, fillet weld design stresses should be at least 1.7 times UBC allowable stresses, rather than 1.33 times UBC. However, since highly stressed welds do lose stiffness, the assumed doubler plate stiffener thickness for drift control should be limited to the fillet leg dimension.

C706.3 UBC §2211.7.3 Flange Width-Thickness Ratio

The additional compact section requirement in regard to flange width-thickness ratio reflects LRFD analyses and is intended to limit local flange buckling.

Girders shall meet the requirements for compact sections contained in AISC §F1.1, with the exception that $b_f/2t_f$ shall meet the requirements contained in AISC §N7 for single web shapes that are subjected to compression involving hinge rotation under ultimate loading. Although Equation C706-1:

$$52/\sqrt{F_y} \qquad \textbf{Eqn. C706-1}$$

gives slightly higher values of $b_f/2t_f$ than contained in AISC §N7. Equation C706-1 was selected because it is satisfied by virtually all rolled beam sizes in A-36 steel, normally used in ductile frame members. Column sections are required to meet the $b/2t_f$ requirements of AISC §N7. Designs should also consider the potential impact of actual yield strengths that are significantly above the specified minimum (see Section C703.1).

C706.4 UBC §2211.7.4 Continuity Plates

Analytical and experimental research results suggest that connections with flexible column flanges lead to high stress concentration in the beam flange-to-column weld near the column web. Stiffeners (or continuity plates) are used to help reduce this stress concentration and prevent premature fracture initiation. However, with jumbo column sections, the benefits of adding stiffener plates may not be quite as pronounced. It should be noted that if reinforced connections are used, the force that can be delivered to the column flange by the beam will be increased, and therefore, the requirements for continuity plates, under the AISC specifications, also increase. More testing is needed in this area before conclusive recommendations can be made either for large sections or for new connections configurations that deliver higher forces into the columns.

Consensus opinion of the SAC project is that full-depth column flange stiffeners (continuity plates) with a minimum thickness equal to 1/2 that of the beam flange should be used in all connections, even when no stiffeners are required (or thinner ones are permitted) by the AISC specification. Fillet welds to the column web were thought to be acceptable, but double bevel complete joint penetration (CJP) welds were recommended at the flanges. Backing bars need not be removed from the continuity plate CJP welds. It should be noted that some SAC workshop participants felt that the continuity plates should be the same thickness as the beam flange in order to ensure a uniform distribution of stress across the width of the column flange. Other participants, however, felt that the use of stiffeners should be minimized. They felt that use of stiffeners results in added restraint and shrinkage stresses on the CJP welds of the beam-to-column flange, and potentially aggravates the potential for column through-thickness fractures.

The AISC specifications address the column flange thickness needed to support a given beam flange force without yielding the column flange in bending about the column web. However, the beam flange force cannot be taken as the seismic design force. The specified design force is that force needed to cause the beam to hinge in flexure.

Tests [Popov et al., 1986; Butler and Kulak, 1971] show that beam hinging occurs at local flange buckling when the flange of a compact section is strain-hardened to about $1.3F_y$. However, the beam flange force at the beam-column connection must strain to about $1.8F_y$ if the entire beam M_p is resisted only by flanges that provide only $0.70M_p(1.3/0.7 = 1.8)$ [Popov, 1987]. This neglects the small contribution of web bolting, the fact that the Z_f/Z ratios can vary, that the spread between F_y and F_u may not be 30 percent, and that the supplied steel strength margin can vary independently for the beam and the column. Since the plate is needed to help protect the beam flange-to-column flange weld from failing due to uneven stress distribution at up to ultimate tensile stress, the flange continuity plates should be provided wherever they *might* be needed. Continuity plates provide a well-defined panel zone for stress analysis and they were provided in almost all joint tests prior to the Northridge earthquake. Panel zone tests have also shown that where the AISC requirements were not satisfied, the joint performed poorly.

Figures C706-1 and C706-2 show detailing concepts related to the panel zone and continuity plates. Section C706.4 uses what, to some engineers, may be a confusing terminology. Technically, the plates referred to in Section C706.4 should be called tension flange "stiffener" plates; while the plates used to "continue" the flanges of girders connected to a column in its weak direction should be called "continuity" plates. This terminology is used in the figures.

The figures represent detail concepts only. The connections shown do not necessarily satisfy the strength requirements discussed in C706.1. The specifics of the figures should not be construed as required, or even recommended, details, except where they represent specific UBC code provisions. Caution should be exercised in providing specific welding details in these highly restrained areas.

C706.5 UBC §2211.7.5 Strength Ratio

Previous Blue Book Requirements were silent on the issue of column hinging, although concerns were expressed in the Commentary. Most engineers believe that it is better to have flexural hinges form in the girders than in columns [Roeder, 1987]. If flexural hinges form in the columns, additional concerns include: concentration of inelastic behavior in the columns of a story leading to a weak story condition; and excessive inelastic deformations leading to column failure under the possible high axial loads (as represented in these Requirements by $3(R_w/8)$ times design loads) that could occur due to maximum expected ground motion response. The strong column/weak girder concept is addressed with the introduction of simple formulas, UBC equations 2211-3.1 and 2211-3.2, which require the potential for yielding at a column-girder joint to be located in the girder or the joint panel zone as a standard, but with exceptions. The plastic section moduli are used in equation 2211-3.1 because they are more representative of girder moment strengths at higher strain levels. When steels of different strengths are specified (e.g., A36 girders and A572 Grade 50 columns), consideration should be given to the relative overstrength levels, as described in C703.1. In addition, the effect of moving the plastic hinge locations away from the column face must be considered in determining the maximum girder moments that can be generated.

> ***Exception*:** UBC equations 2211-3.1 and 2211-3.2 permit the potential for yielding to be located in the column, adjacent to the joint, if certain beam limitations for compact shapes (Section C706.3) are also applied to the column;

and, in addition, if the column compressive stress is limited to f_a less than $0.4F_y$ for all load combinations, except as specified in Section C704.1.

Although the latter exception allows the weak column to have f_a larger than $0.4F_y$ due to the load combinations specified in Section C704.1, consideration should be given to maintaining f_a less than $0.4F_y$ for these Section C704.1 combinations. This would serve to provide the needed capacity for columns that are required for stability under the maximum load conditions in the inelastic structure.

These limitations, along with provisions for joint support in Section C706.7, are intended to provide stability conditions under which a column may have limited yielding. Exceptions b and c exclude columns that either do not have a potential for yielding due to their large lateral story shear capacity relative to the story above, or are not counted on to contribute to the story shear capacity at all. It is to be strongly emphasized that this exception is given for special and unique design situations where it would be highly impractical or infeasible to meet the requirement of UBC equations 2211-3.1 and 2211-3.2.

This exception is to be used with caution. It should be limited to the minimum number of columns in any frame system, but may be particularly applicable to weak axis column conditions in highly redundant "general" moment frames. The majority of columns in any story should conform to UBC Eqns. 2211-3.1 and 2211-3.2. This exception should not be used to create systems that would have a predominance of weak columns in any story or portion of the total structure. Tests on moment frame subassemblages [Krawinkler, 1971; Zayas et al., 1981], in which yielding of columns occurred, did not exhibit any loss of lateral force resistance at displacements representative of maximum expected earthquake response; however, one test column [Zayas et al., 1981] meeting the requirements of the exception, exhibited local web and flange buckling and was unable to properly support its load [Roeder, 1987].

C706.6 UBC §2211.7.6 Trusses In SMRF

The use of ductile frames made up of columns and trusses has been recognized. The trusses carry gravity loads and also function as the horizontal "girder" members of the lateral force resisting system. Examples where this type of framing may be employed are: for hospital buildings where a deep "utility" space is required above the ceiling level; and in one-story industrial buildings that have long spans between columns. Since trusses are made up of axially loaded members without provisions for flexural ductility, it is the columns that need to yield in flexure to protect the truss. Therefore, such columns need special provisions to ensure their ductility, joint stability, and resistance to torsion. The Requirements of this section are intended to provide the necessary constraints for this type of frame. The truss structure and connection must have a strength sufficient to force the columns to hinge. Note that such columns do not meet the intent of UBC equations 2211-3.1 and 2211-3.2 and must, therefore, qualify for one of the cases in the exception in Section C706.5.

Although not specifically mentioned in Section C706.6, it is not intended that the load combinations of Section C704.1 be carried within the $0.4F_y$ axial stress limit.

Referring to Commentary Section C706.5 with respect to frames having the weak column-strong girder condition, many engineers strongly believe that the truss girder frame system described above should not be classed as a special moment resisting frame with the $R_w=12$ value. The amount of inelastic energy dissipation capability for the truss girder system may be more appropriately represented by the $R_w=6$ value used for the ordinary moment resisting steel frame.

A recent research program carried out at The University of Michigan has shown that truss moment frames designed as ordinary moment frames ($R_w=6$) possess rather poor hysteretic behavior, with large abrupt drops in strength and stiffness due to buckling and early fracture of web members. Such degrading hysteretic behavior can result in excessively large story drifts of the building frames when subjected to earthquake ground motions with peak accelerations of the order of 0.4g to 0.5g [Itani et al., 1991; Goel et al., 1994].

This research led to the development of a new design concept that ensures stable hysteretic behavior of the truss girders [Goel et al., 1994b; Basha et al., 1994]. The system is called Special Truss Moment Frame (STMF) and the design provisions are contained in Section 2211.12 of the 1996 Accumulative Supplement of the UBC with an R_w of 10. A revised and more complete version of the provisions has been approved by the ICBO Code Development Committee for 1997 UBC. The design of STMF aims at limiting inelastic deformations in the event of extreme loading to occur in selected segments of the truss girders. The chords and web members (arranged in X-pattern) of the special ductile segment are designed to withstand large cyclic deformations, while the rest of the structure remains elastic. The result is a truly "strong column-ductile girder" combination with stable hysteretic behavior. The design concept and procedure have been validated by extensive testing and analytical work.

C706.7 UBC §2211.7.7 Girder-Column Joint Restraint

Moment frames should be capable of reaching their seismic design capacity without failure due to frame lateral torsional buckling. The column needs to be well braced at the beam-column joint to prevent column rotation out of the plane of the moment frame.

If the column flanges at a restrained joint remain elastic, they need be braced only at the level of the top flange. UBC §2211.7.7.1 defines four conditions where columns may be assumed to remain elastic. This criterion, even though arbitrary for lack of test data, was felt necessary by SEAOC. The condition (UBC §2211.7.7.1.4) may be interpreted as having the column stress ($f_a + f_{bx} + f_{by}$) limited to F_y. If the columns are permitted to yield at the joint, then it is necessary to brace the column flanges at both the top and bottom of the joint.

SEAOC suggests for this lateral support criterion a brace capable of resisting a force equal to one percent of the girder flange strength (F_yA_f), while limiting the maximum displacement perpendicular to the frame to 0.2 inch. Definitive bracing criteria explicitly related to the repeated short-term nonlinear excursions associated with building response in earthquakes have not yet been verified with tests. Nevertheless, from the data available, a fairly consistent criterion can be defined.

In developing the 1985 Blue Book, SEAOC at first considered two exact numbers, one percent and 0.1 inch, for bracing design, only to relax the stiffness requirement to 0.2 inch. This change is justifiable on the basis that the 0.1-inch limitation is based on beams subject to a constant moment, which is a very conservative situation when compared to the typical condition design case for seismic moment frames that have a moment gradient in the beams.

Section C706.7.2 addresses those situations where the girder-column joint is unrestrained, such as in two-story frames at mechanical equipment floors. It is extremely important that the columns do not buckle out of the plane of the seismic frame. To ensure this, four design requirements are imposed on columns without lateral support transverse to the joint: 1) increased seismic forces; 2) reduced allowable axial stress; 3) a reduced column slenderness ratio; and 4) the inclusion of the effects of the bracing and P-delta forces when figuring bending stress about the weak axis of the column.

C706.8 UBC §2211.7.8 Beam Bracing

Beam bracing for the reversed bending moment involved in a seismic moment frame can usually be provided by the intersecting beams, which usually are spaced a maximum of 12 feet on center. Typical moment frame girders have an r_y of at least 1.5 which, when combined with 12-foot beam (brace) spacing, translates into a girder bracing limitation of $L_{cr} = 96r_y$.

The code allows such bracing to be "direct or indirect." Direct bracing would include a perpendicular element that is directly attached to the unbraced flange, such as in a diagonal "kicker" brace. An example of indirect bracing is the use beam web stiffeners that are welded to the unsupported flange and deliver the bracing loads to a shallower perpendicular floor beam. The stiffness of such indirect bracing should be considered to verify their adequacy.

C706.9 UBC §2211.7.9 Changes in Beam Flange Area

This Provision is intended to prevent concentration of beam inelastic strain at abruptly reinforced or reduced beam sections.

Note that since the Northridge Earthquake, interest has been generated in so-called "dog-bone" connections where the beam flanges are intentionally reduced in area in an effort to limit the demand on the welded connections and provide a defined region for inelastic action. These connections, which generally have a rather gradual change in flange area, should be considered as satisfying this provision.

C706.10 UBC §2211.7.10 Moment Frame Drift Calculations

Drift calculations are important in regard to P-delta control, distortion control, and dynamic response. This section provides needed definition for these calculations.

Significant contributions to drift can result from axial deformations of columns in taller structures, and from shear distortion of column webs in joint panel zones (as well as from flexural and shear deformations of the clear spans of the frame elements). These Recommendations require that contributions from all of these effects be included in drift calculations unless their contributions are small.

In order for axial deformations to be neglected, their contribution must be less than 10 percent of the total story drift. An acceptable approach to demonstrating that this condition is satisfied is to use an example frame of similar configuration and member size as a basis for calculation.

The shear distortion in column panel zones must be included in drift calculations unless the panel zone has sufficient strength to develop 80 percent of the flexural strengths of girders framing into the column.

C707 Requirements for Braced Frames

C707.1 UBC §2211.8.1 General

In these Requirements braced frames have been defined as a truss system or its equivalent that is provided to resist lateral forces in the frame system and in which the members are subjected primarily to axial stress. Chapter 7 places the horizontal "girder" truss-column system, where the truss acts as a beam in a flexural frame, in the moment resisting frame Section C706.6.

Braced frames are then divided into three basic types, concentric, special concentric, and eccentric braced frames. This section addresses concentrically braced frames, which include diagonal and cross-braced configurations, along with two special classifications, "chevron" bracing and "K" bracing (Figure C707-1).

With this edition of the Blue Book, an additional set of requirements has been added to the concentrically braced frame requirements of the UBC to improve the inelastic response and increase the ductility of these frames. This newly defined system is called a "special concentrically braced frames" (SCBF), and is addressed in UBC §2211.9.

Eccentrically braced frames, as shown in Figure C707-2, have a new definition and set of design requirements in Section C708. The eccentrically braced frame (EBF) is credited with ductility similar to that of special moment resisting frames (SMRFs), and has many special provisions intended to ensure that ductility.

C707.1.1 ***Ductility Demands.*** Ductility by means of reversible inelastic lateral distortions is the basic justification for seismic design based on elastic response forces factored down from those expected in a fully elastic structure during a strong earthquake. Concentrically braced frames (CBFs) may exhibit severely limited reversible inelastic distortions. Inelastic axial stretching and shortening of slender members is not reversible. Inelastic reversible axial stretching and shortening of other compression members is only possible up until significant compression buckling takes place. Tests [Popov and Black, 1981; Jain et al., 1980] show that after buckling, an axially loaded member loses strength rapidly with repeated inelastic load cycles and does not return to its original straight-line position.

It is for these reasons that concentrically braced frames have not been allowed by code for tall or special buildings without the backup of a ductile moment frame. In addition to this limitation, the basic design force for concentrically braced frame structures was set at 1.5 times that of a ductile moment frame system, and in the past an additional 1.25 factor was applied to brace design. This was done to raise the level of elastic force design to provide for low to moderate intensity earthquakes that are most likely to occur.

C707.1.2 ***Expected Performance of CBFs.*** The performance of CBFs is expected to be very good as long as the frame remains elastic, local buckling fracture of brace member elements does not take place, and connections do not fail. The emphasis of these Requirements is therefore on raising the elastic design level to cover most earthquake exposures.

When an axially loaded brace buckles in compression, several things can happen:

1. Capacity of the buckling brace drops, causing the tension-acting brace to take more load.
2. Buckling of the member may cause excessive rotation at the ends of the brace, and could lead to local connection failure.
3. Buckling can cause local or torsional buckling near midspan of the brace.
4. Buckling in the brace can cause nonstructural failure in the brace encasement.
5. Residual building displacement can result.
6. If brace buckling occurs nonsymmetrically, torsional response can be induced.
7. The brace buckling results in a significant loss of stiffness during cyclic load reversals, which could lead to the development of large velocities prior to the straightening and sudden development of tension in the brace.
8. Excessive buckling can affect nonstructural systems attached to the frame.

Low L/r ratios and low stresses will reduce or prevent brace buckling for small or moderate events; this is the emphasis of these CBF Requirements. However, the potential for limited ductility and poor inelastic response of these frames in strong earthquakes, as manifested by rotation of brace end connections and local buckling fracture of brace elements (tube walls and stitched angles) must be recognized by the engineer. Serious consideration should be given to incorporating the Requirements of Section C708 for SCBFs whenever feasible.

C707.2 Bracing Members

C707.2.1 ***UBC §2211.8.2.1 Slenderness.*** For structures taller than two stories, the slenderness ratio L/r (not KL/r) of braces is limited to a maximum value of $720\sqrt{F_y}$. The following argument is used for this provision.

In the postbuckling range, the compressive strength of axially loaded members deteriorates and the hysteresis loops take on a severely pinched shape (Figure C707-3). Braces with small L/r will dissipate more energy because in the postbuckling range they will undergo cyclic inelastic bending, which slender braces will not. Very slender braces have almost no stiffness while they straighten out from a buckled configuration and in the straightened configuration (tension) pick up stiffness very rapidly. For a braced frame, this rapid increase in stiffness may cause impact loading during cyclic loading. This impact may overstress the brace or cause a brittle failure of the brace connections.

Unfortunately, the choice of a brace with a small L/r is not always ideal either. Such braces undergo cyclic inelastic bending in the postbuckling range. The curvatures associated with this cyclic inelastic bending may be very large and local buckling has to be expected. This local buckling may be severe and may lead to localized kinking of plate elements which in turn may cause crack propagation and fracture. Such fracture has been observed rather early in tests of tubular bracing members. This characteristic is likely more prevalent in square tube braces. Tube sections may be selected to have conservative (low) wall width-thickness ratios so as to minimize the effects of localized buckling. Alternatively, consideration should be given to other methods of wall stiffening such as filling tubes with concrete [Goel et al., 1992c].

Single angle bracing members should be avoided, especially in compression, since it is virtually impossible to load single angles concentrically, and eccentric loading normally leads to large bending stresses, which are difficult to account for properly in design.

C707.2.2 ***UBC §2211.8.2.2 Stress Reduction.*** According to these Requirements, the UBC code-allowable stress for axially loaded braces in CBFs must be decreased by a stress reduction factor, β, which accounts for deterioration under cyclic earthquake loading. The value of this stress reduction factor is determined from UBC equation 2211-5. This relationship is felt to provide an approximation of the strength reduction observed in published [Popov and Black, 1981; Jain et al., 1980] hysteresis loops for tested axial members after a reasonable number of cycles. It can be shown that use of this factor is equivalent to use of a force multiplier varying from 1.0 for $KL/r = 0$ to 1.5 for $KL/r = C_c$. Thus, for very short braces, it imposes a smaller penalty than the 1.25 factor used in previous codes, while for longer braces it is more severe.

When evaluating bracing system strength for purposes of computing the maximum load it can impose on other systems (such as columns, see Section C704.1), the cyclic load factor should not be used, since this would lead to underestimation of bracing system strength for the early cycles of seismic response.

> *Exception*: Bracing members can be designed using the requirements for column strength when they carry gravity loads. Caution should be exercised in implementing this exception for conditions where the gravity loads are small in comparison to the seismic effects.

C707.2.3 ***UBC §2211.8.2.3 Lateral Force Distribution.*** Nearly equal strength bracing in opposite directions is required on every line of bracing (or at least on closely adjacent lines) to prevent an accumulation of inelastic deformations in one direction only.

The exception in UBC §2211.8.2.3 is provided since it should be acceptable to have all or most of the bracing in one direction if it is of sufficient strength to preclude buckling except under the most severe of loadings. The $3(R_w/8)$ times code has been used throughout these Requirements as an estimate of a "severe loading."

C707.2.4 ***UBC §2211.8.2.4 Built-Up Members.*** The integrity of a brace in the postbuckling range requires that buckling of individual segments between stitches be prevented. Considering that the buckled brace may take on a significant curvature, and that end restraints are often neglected in the calculation of KL/r for the brace, it is necessary to reduce the slenderness ratio of individual segments to a value lower than that for the brace as a whole. Note that the reference here is to L/r rather than KL/r. Rigid shear transfer must be provided across stitches so that the shears associated with the curvature in the buckled brace can be transferred across the stitch without slip. Welded stitches are recommended. Machine bolted stitches should not be used. The AISC seismic provisions recommend that the stitches be designed for a force equal to 50 percent of the nominal strength of the individual members [AISC, 1992].

C707.2.5 ***UBC §2211.8.2.5 Compression Elements in Beams.*** Appendix C of the AISC specification provides for reduction of allowable stresses for elements that exceed the permitted width-thickness ratios; however, it should not be applied to elements of members expected to exceed allowable stresses. Provision of braces meeting the more restrictive width-thickness ratios for rectangular tube members, such as those required for SCBFs, should be considered to improve the inelastic response of these concentrically braced frames. Note that the 1997 UBC has changed this section to be identical to Section 2211.9.2.4 for special concentrically braced frames.

C707.3 Bracing Connection

C707.3.1 ***UBC §2211.8.3.1 Forces.*** In braced frames, the bracing members must provide most of the lateral resistance, particularly if no backup frame exists. In a major earthquake, the force demand on a brace in tension is expected to exceed the brace tensile yield strength as defined in Section C703.2. In order to avoid brittle failure at the brace connections, the connections must be designed to develop the tensile yield strength of the brace, or at least the maximum force that can be delivered to the system or at least the force level associated with a severe loading: $3(R_w/8)$ times prescribed seismic forces. These same minimum strength requirements apply also to beam connections that are part of the bracing system.

An additional requirement has been added to ensure that allowable stresses on the bracing connections are consistent with brace design forces. Beam-column connections that are part of the braced frame must also be checked to ensure they can deliver the bracing connection design forces. Proper consideration of connection eccentricity is explicitly required to verify completion of the load path.

C707.3.2 ***UBC §2211.8.3.2 Net Area.*** For welded brace connections, design is straightforward because no reduction of section is necessary. For bolted connections, the provisions of the LRFD specifications [AISC, 1986] are prescribed.

C707.4 Bracing Configuration

C707.4.1 ***UBC §2211.8.4.1 Chevron Bracing.*** Frames with this type of bracing may exhibit two problems that set them apart from braced frames in which both ends of the braces frame into beam-column joints. First, if one of the braces buckles, large vertical deflections will occur in the floor system at the connection within the beam span (Figure C707-4). Second, the postbuckling strength of the bracing system may deteriorate significantly because the tensile force that can be developed in the tension brace can exceed the decreasing postbuckling strength of the compression brace only by the additional force necessary to cause hinging of the floor beam. This additional force will be small unless the beam is of substantial size. Also, once a brace buckles it will not straighten out fully under load reversal, the opposite brace will buckle at load reversal and the story shear resistance of the bracing system will be governed by the postbuckling behavior of two opposed braces. The consequence will be a deteriorating story shear-story drift relationship (Figure C707-5). The only reason the relationship shown in Figure C707-5 does not exhibit more rapid deterioration is that in the test structure, from which this relationship was obtained, the chevron bracing system was surrounded by a ductile moment resisting frame that provided a significant part of the shear resistance once the bracing buckled.

SEAOC has considered a number of options for chevron-braced frames, ranging from prohibition, to more stringent limitations on slenderness ratios, to increased design load levels for the diagonal bracing members. The third option was finally selected and the seismic design loads for the diagonal bracing members were increased by a factor of 1.5. This appears to be a considerable increase, but in the opinion of many committee members is barely adequate. The cyclic load factor, β, discussed in UBC §2211.8.2.2, is certainly helpful since it will lead to a substantial further increase in lateral load resistance if slender braces are used. UBC §§2211.8.4.1.2 and .3 were introduced to require some minimal strength in the beam element to resist gravity loads in the event of a loss in brace capacity. The UBC exception on beam strength for roof stories, penthouses, and one-story structures is provided since large deformations of the roof may not have life safety consequences as they might for floors. The 1.5 factor is considered to be a sufficient strength increase for these structures without the beam strength requirement. The AISC seismic provisions also require the provision of lateral support at the intersection of chevron braces.

C707.4.2 ***UBC §2211.8.4.2 K-Bracing.*** The K-bracing configuration is undesirable for seismic resistance. Deformations of the bracing system due to tensile yield and/or buckling may cause lateral deformations of the connected column sufficient to cause column buckling and subsequent collapse.

Where seismic forces are low and improbable, as in Seismic Zone 2, or for smaller less important structures designed for high lateral loads, this prohibition is felt to be too restrictive. However, K-bracing is not recommended, even where it is permitted, except where other configurations are impractical.

C707.4.3 ***UBC §2211.8.4.3 Nonconcentric Bracing.*** Where minor eccentricities (defined as being less than the depth of the affected member) are incorporated in the CBF configuration, this Section applies. The eccentricity must be properly considered in the design, in accordance with the required bracing connection forces. Creation of such eccentricities in building columns is not recommended, for the same reasons as those described for K-bracing.

C707.5 UBC §2211.8.5 One- and Two-Story Buildings

For smaller less important buildings, the provisions of UBC §2211.8.2 and §2211.8.4 may be waived if the structure has the strength to resist $3(R_w/8)$ times prescribed seismic forces. This, for example, would permit tension-only bracing for such structures.

C708 Special Concentrically Braced Frames (SCBF)

C708.1 UBC §2211.9.1 General

Steel CBFs have traditionally been among the most efficient structural systems for resisting lateral forces. Since their adoption into seismic design codes, improvements have been made to CBFs with emphasis on increasing brace strength and stiffness, primarily through the use of higher design forces that would minimize inelastic demand. More recently, ductility as an essential ingredient distinguishing lateral force resisting systems in seismically active areas, has been applied to the design of CBFs.

Actual building earthquake damage (including the 1994 Northridge earthquake), as well as damaged observed in laboratory tests of CBFs have generally been caused by limited ductility and brittle failures. These brittle failures are most often observed as fracture of connection elements or brace members. Lack of compactness in braces results in severe local buckling of the brace, which leads to high concentrations of flexural strains at these locations, and reduces their ductility. CBFs are subject to severe local buckling, with diminished effectiveness in the nonlinear range, at low story drifts. The large story drifts that result from brace fractures can impose excessive ductility demands on the beams and columns, or their connections.

Research conducted over the past 15 years at the University of Michigan has demonstrated that CBFs, with proper configuration, member design, and detailing can possess ductility far in excess of that previously ascribed to such systems. Extensive analytical and experimental work performed by Professor Subhash C. Goel and his collaborators has shown that improved design parameters, such as limiting width/thickness ratios (to prevent local buckling), closer spacing of stitches, and special design and detailing of end connections greatly improve postbuckling behavior of CBFs. A new system reflecting these developments, referred to as special concentrically braced frames (SCBFs), has been added to these Requirements.

Previous requirements for CBFs sought to ensure reliable behavior by limiting global buckling. Cyclic testing of diagonal bracing systems verifies that energy can be dissipated after the onset of global buckling if brittle failures due to local buckling, stability problems, and connection fracture are prevented. When properly detailed for ductility, as prescribed in these Requirements, diagonal braces can sustain large inelastic cyclic deformations without experiencing premature failure.

Analytical studies [Tang et al., 1987; Hassan et al., 1991] on bracing systems designed in strict accordance with earlier code requirements for CBFs predicted frame failures without the development of significant energy dissipation. Failures occurred most often at plastic hinges (local buckling due to lack of compactness) or in the connections. Plastic hinges normally occur at the ends of the brace and at the brace midspan. Analytical models of bracing systems designed to UBC SCBF provisions, when subjected to the same ground motion records as the previous CBF designs, exhibited full and stable hysteretic loops without fracture. Similar results were observed in full scale tests conducted as part of the "U.S.-Japan Cooperative Earthquake Research Program" [Wallace et al., 1985; Tang et al., 1989].

For double angle and double channel braces, closer stitch spacing, in addition to more stringent compactness criteria, are required to achieve improved ductility and energy dissipation. This is especially critical for double angle and channel braces that buckle in such a way that large shear forces are imposed on the stitches (Figure C708-1). Studies also showed that placement of double angles in a toe-to-toe configuration reduce bending strains and local buckling [Goel et al., 1992c].

Many of the failures reported in CBF systems due to strong ground motion have been in the connections. Similarly, cyclic testing of specimens designed and detailed in accordance with typical CBF provisions have produced connection failures [Astaneh et al., 1986]. Typical design practice has been to design connections only for axial loads. Good postbuckling response demands that eccentricities be accounted for in the connection design, which should be based on the maximum forces the connection may be required to resist. Studies conducted at the University of Michigan suggest that good connection performance can be expected if the effects of brace member cyclic postbuckling behavior are considered [Goel, 1992a].

For brace buckling in the plane of the gusset plates, the end connection should be designed for the full axial load and end moment capacity of the brace [Astaneh et al., 1986]. Note that a realistic value of "K" should be used to represent the connection fixity.

For brace buckling out of the plane of single plate gussets, weak axis bending in the gusset is induced by member end rotations. This results in pin-like end conditions with plastic hinges at midspan in addition to the hinges that form in the gusset plate. Satisfactory performance can be ensured by allowing the gusset plate to develop restraint-free plastic rotations. This requires that the free length between the end of the brace and the assumed line of restraint for the gusset be sufficiently long to permit the plastic rotations and short enough to preclude the occurrence of plate buckling prior to member buckling. A length of two times the plate thickness is the recommended standard [Astaneh et al., 1991] (Figure C708-2). As an alternate to permitting plastic hinging in the connection plate, connections with stiffness in two directions can be detailed. Crossed gusset plates are an example of such a connection. Test results indicate that forcing the plastic hinge to occur in the brace rather than the connection plate results in greater energy dissipation capacity [Lee et al., 1990].

Since the stringent design and detailing requirements for SCBFs will produce more reliable performance when they are subjected to the high energy demands imposed by severe earthquakes, the design force level has been reduced below that required for CBFs. This has been accomplished both by raising the R_w value and by eliminating the β amplification factor.

Bracing connections should not be configured in such a way that beams or columns of the frame are interrupted to allow for a continuous brace element. This provision is necessary to improve the out-of-plane stability of the bracing system at these connections.

Horizontal bracing that transfers forces between horizontally offset bracing in the vertical plane, if classified as a SCBF system, must comply with these Requirements. Such transfer bracing is exempted, however, from the requirements for closer stitch spacing and from the unbalanced force design in chevron configurations. In addition, they are not subject to the K brace prohibition.

> ***Exception***: Horizontal diaphragms composed of diagonal elements are exempt from these Requirements. It is reasoned that diaphragms will not normally be required to undergo extreme nonlinear stress reversals and therefore may be designed in accordance with UBC Standard 27-15.

C708.2 UBC §2211.9.2 Bracing Members

The maximum slenderness ratio (KL/r) has been set at $1000\sqrt{F_y}$. Studies have shown that brace member postbuckling cyclic fracture life often increases as the KL/r ratio increases [Tang et al., 1989]. It was felt, however, that some limit should be imposed to prevent buckling during low to moderate ground motions. Since good inelastic response is not inhibited by global buckling, a lower limit on slenderness did not seem warranted.

The requirement that bracing members along any bracing line be split between compression and tension members was carried over from earlier CBF provisions. This is intended to prevent an accumulation of inelastic deformations in one direction.

Closer stitch spacing than required in previous code provisions is needed to restrict individual element bending and premature cracking in brace members [Xu et al., 1990].

> *Exception*: Wider spacing is permitted when buckling will not cause shear in the stitches.

Width-thickness ratios of compression elements in brace members have been reduced to be at or below the requirements for compact sections to eliminate the occurrence of premature local buckling and subsequent cracking of brace elements during repeated inelastic cycles. Tests have shown this failure mode to be especially prevalent in rectangular tube elements with width-thickness ratios larger than the prescribed limit [Tang et al., 1989].

C708.3 UBC §2211.9.3 Bracing Connections

UBC §2211.9.3.1 (Forces) and 2211.9.3.2 (Net Area) are carryovers from prior codes. Their purpose is to prevent brittle failure at the brace connection. The full expected tension force would be at least P_{st}. These minimum strength requirements apply also to other connections that are part of the bracing system, including the connection to the foundation system.

When brace buckling is in the plane of a gusset plate, the plate should provide flexural strength that exceeds that of the brace. This will ensure that plastic hinges will form in the brace, rather than in the plate.

When brace buckling is out of plane of a gusset plate, the length needed to allow restraint free plastic rotation is twice the plate thickness. This mode of buckling may lead to preferable inelastic behavior (Figure C708-2) [Popov and Black, 1981; Astaneh et al., 1986].

C708.4 UBC §2211.9.4 Bracing Configuration

Chevron brace requirements for CBFs require a 1.5 amplification of the brace design force to account for effects such as the unbalanced force between a fully effective tension brace and a compression brace that has lost much of its capacity in the postbuckling state. These SCBF provisions require that the beam must possess sufficient strength to resist this potentially very significant postbuckling force redistribution (the unbalanced force) in addition to all other forces to which the beam will be subjected (Figure C707-4). Thus the 1.5 factor is not needed. Hysteretic loops from tests of typical bracing members demonstrate a residual postbuckling compression capacity of about 30 percent of the original compression strength. This is the maximum magnitude of compression force that should be combined with the full tension force for design of the beam. The full tension force can be expected to be in the range of P_{st}.

K-bracing is generally prohibited in the CBF requirements and is totally prohibited in these SCBF requirements. It is considered undesirable to subject a column to brace forces as these forces may contribute to column buckling.

C708.5 UBC §2211.9.5 Columns

When subjected to severe ground shaking, columns in braced frames can experience significant inelastic shear and bending after buckling and yielding of the braces. Therefore, columns must have adequate compactness, as well as sufficient shear and moment capacities to allow them to maintain their lateral strength during nonlinear cyclic deformation of the frame. In fact, analytical research on SCBFs without backup SMRFs have shown that columns carry as much as 40 percent of the story shear [Tang et al., 1987; Hassan and Goel, 1991]. When columns are common to both SMRFs and SCBFs, their portion of the story shear may be as high as 50 percent. SCBF column splice requirements for shear are more restrictive than those for SMRFs.

C708.6 UBC §2211.9.6 Nonbuilding Structures

As with CBFs some of the requirements are waived for SCBF nonbuilding structures.

C709 Eccentrically Braced Frame Requirements

C709.1 UBC §2211.10.1 General

Prior to the 1988 UBC the eccentrically braced frame was only implicitly covered by the code definition of braced frames, because there were no special code provisions that addressed EBFs. The very limited use of EBFs was under the use limitations and seismic force requirements of "braced frames," with normal elastic design. Research has shown that buildings using the EBF system have the ability to combine high stiffness in the elastic range with excellent ductility and energy dissipation in the inelastic range. In the elastic range, the lateral stiffness of an EBF system is comparable to that of a CBF system, particularly when short link lengths are used. In the inelastic range, EBF systems can provide stable, ductile behavior under severe cyclic loading, comparable to that of a properly detailed SMRF system. UBC §1625 gives this EBF frame an independent classification with fewer use limitations and reduced design forces, to be used with a set of very detailed requirements detailed in this Section C709.7. The detailed requirements are intended to ensure good inelastic cyclic performance.

An eccentrically braced frame is a braced frame with at least one end of each brace intersecting a beam, eccentric to the beam intersection with the column or the opposing brace. The short section of the beam between opposing braces, or between a brace and the beam-column intersection is called the "link beam" and is the element of the frame counted on to provide inelastic cyclic yielding.

The intent of the eccentrically braced frame system is to provide a ductile link where the structural system will yield in lieu of buckling of its braces, when the frame experiences dynamic loads in excess of its elastic strength. To ensure, and take advantage of, the ductility of the links, it is important that all framing elements applying forces to the links and resisting those forces be strong enough to force the links to yield; and that these elements maintain their integrity through the range of forces and displacements developed during the hysteretic action of the links. The braces, beams outside the links, columns and collector beams are designed to generally remain elastic throughout the range of response of the links. Therefore, additional strength requirements are imposed on braces, beams outside of the links, and, to a lesser extent, on columns, as described in Sections C709.13 and C709.14. Collector strength recommendations are described in Section C709.17.

The rotation demand on the link beam is a multiple of the lateral drift of the frame as a whole; which is a function of the geometry of the frame (Figure C709-1). Since short link beams that yield in shear are considered to have the most stable energy dissipation characteristics, most of the EBF research at the University of California at Berkeley tested the cyclic inelastic capacity of link beams with shear yielding at large rotations. Most of the Requirements in this Section are concerned with limiting the link beam shear yield rotation to less than the limiting test rotations and requiring details that tests have indicated are necessary to ensure the required ductile rotation capacity.

It has been observed in moment frame panel zone tests that shear yielding is very ductile and that the inelastic shear force capacity based only on beam web shear area greatly underestimates the web plus flange inelastic shear capacity. However, the link beam shear yield rotation demands can be much greater than panel zone demands and the panel width-to-depth ratios are different. It was found that where the ratio of the sum of the depth plus width dimensions of the beam panel to the web thickness exceeds about 90, web buckling reduces the beam inelastic cyclic shear capacity. Therefore, based on tests at University of California at Berkeley [Hjelmstad and Popov, 1983], several Requirements specify requirements for web stiffeners (see Sections C709.7 through C709.11). Section C709.4 limits link beam rotation. Sections C709.1, C709.3, C709.5, and C709.17 relate to EBF beams. Connection requirements are covered in Sections C709.6, C709.12, and C709.19. Brace beam beyond the link design is given in Section C709.13. Column design at beam-column joints is given in Section C709.14. Concentrically braced frames are permitted at the roof (Section C709.15) and at the first story with a strength margin to ensure that the braces will not yield (Section C709.16). Lateral bracing of the beams is covered in Section C709.18.

Test results confirm the validity of the capacity design approach for the braces, the beams outside the links, and the columns. This approach involves the design of these elements for force levels consistent with ultimate shear or moment strength of the link. Ultimate strength level forces include link overstrength due to factors such as strain hardening, actual yield strength of the link exceeding the specified yield strength, or effects from a composite floor system.

The ratio of axial force in the brace and in the beam outside the link to that in the link remains essentially constant throughout EBF loading history. This linear relationship remains constant despite link or beam yielding, or beam instability. Therefore, an elastic analysis may be used to estimate the ultimate design forces in the brace and in the beam outside the link. Note that column design moments for conditions with an adjacent link beam requires consideration of the moment distribution in the inelastic state since elastic analyses can provide results that inaccurately reflect distribution of moment to these columns. The capacity design principal requires that the design forces for the brace and for the beam outside the link be increased by a factor of 1.5, as the recommended ratio of ultimate strength of a link to its strength, as defined in Section C703.2.

Experimental evidence suggests that after adequate plastification, the moments at each end of link beams designed and detailed in accordance with the latest code provisions are essentially equal. This assumption of moment equalization at link ends permits application of the following basic static equations:

If shear controls, then:

$$V_{ult} = 1.5V_s \text{ and } \mathrm{M}_{ult} = eV_{ult}/2 \qquad \textbf{Eqn. C709-1}$$

If flexure controls, then:

$$M_{ult} = 1.5M_s \text{ and } \mathrm{V}_{ult} = 2M_{ult}/e \qquad \textbf{Eqn. C709-2}$$

In general, M-V interaction may be neglected in the design of links. While it can have a significant impact on the ultimate shear and moment in intermediate links, the 1.5 overstrength factor is sufficiently conservative in this length range to compensate for neglecting M-V interaction.

Link end moments are shared by both the brace and beam outside the link. In fact, superior brace and beam performance has been achieved experimentally when braces with large flexural stiffness resisted a major portion of the link end moment. The code now encourages this sharing of link end moments by requiring that the brace and beam outside the link have a capacity to develop the 1.5 link overstrength factor (UBC §2211.10.13). It should be noted that this factor was reduced to 1.3 in the 1996 UBC Accumulative Supplement in recognition that some limited yielding in the members beyond the link should not be detrimental to the performance.

Excessive yielding outside the link can result in nonductile failure modes such as severe flange buckling in the brace connection panel. However, limited yielding in the immediate zones outside the link can significantly contribute to the overall energy dissipation capacity of the EBF. Further, this yielding may reduce inelastic deformation demands on the link, which can be particularly beneficial for EBFs with long links.

Arrangements of braces can be made in which links may not be fully effective. One such arrangement is when the links are provided at each end of the brace (e* in Figure C707-2, for example). If the upper link has significantly lower design shear strength than the story below, the upper link deforms inelastically and limits the force that can be delivered to the brace to deform the lower link inelastically. When this condition occurs, the upper link is termed as active link, whereas the lower link is an inactive link. Having potentially inactive links in the EBF system increases the difficulty of analysis. The cost of the structure increases by requiring full link details on the inactive links, even though the brace would be sized by the strength of the active link and the brace connection at an inactive link could be designed as a pin. Thus, it is beneficial to arrange a system that contains only active links.

Most of the detailed EBF provisions have been developed by SEAOC, based on the research of Professor Popov of University of California at Berkeley, and his collaborators [Engelhardt and Popov, 1989; Popov et al., 1987].

C709.2 UBC §2211.10.2 Link Beam

Link beams are the fuses of the eccentrically braced structural system that are to be placed at locations that will preclude buckling of the braces or excessive inelastic response of other elements of the system. A link beam must be located, at a minimum, at one end of each brace.

Compact sections meeting the more restrictive flange width to thickness ratio of $52/\sqrt{F_y}$ are required for the beam portions of eccentrically braced frames in order to provide the beams with stable inelastic deformation characteristics. The same requirement is used for the beams of special moment resisting frames. As noted in Section C703.1, consideration of the variation in material overstrengths should be considered in the design.

C709.3 UBC §2211.10.3 Link Beam Strength

This Provision addresses the concern for the effect that substantial axial loads in the link beam could have on the link beam's inelastic deflection performance. It presumes that the web's capacity is fully utilized in shear and that reliance should be placed only on the flanges for axial and flexural capacity. Links with a length less than approximately 2 M_s/V_s are considered to be controlled by shear.

Substantial axial loads within the link beam can occur when the link beam is required to transmit horizontal forces to or from the braces. Only a small number of link beams with significant axial forces have been tested [Kasat and Popov, 1986]. It is recommended that this condition be avoided by careful selection of EBF configuration and placement within the structure so that the link beams are not required to transmit the horizontal force component of braces or drag struts. The f_a used in UBC §2211.10.3 should correspond to the lesser value of the force required to either yield the link beam in shear or cause the beam flanges to yield. Note that when the axial load in a link beam becomes large ($f_a > 0.15\ F_y$), consideration should also be given to reducing both the design shear strength and the link lengths in order to ensure stable yielding.

C709.4 UBC §2211.10.4 Link Beam Rotation

Link beams in which the link length is 1.6 M_S/V_S or less are classified as short links. In this region, postelastic behavior is dominated by inelastic web shear strains distributed uniformly over the length of the link. These short links have been thoroughly tested and their behavior is well documented. They are capable of sustaining high levels of postelastic end rotation.

Link lengths of 3.0 M_S/V_S or greater are classified as long links. In this case inelastic behavior is dominated by large inelastic flexural strains concentrated at the link ends, with the remainder of the link remaining essentially elastic. Relatively little experimental work has been done with long links. The limited research that has been carried out reveals a tendency for fracture to take place at the link-to-column connection at low levels of flexural rotation. Previous editions of these Requirements used 2.6 M/V as the link length at which "long link" behavior should be expected. Recent research [Engelhardt and Popov, 1989] suggests that 3.0 M/V more nearly approximates this point.

Link lengths between the limits established for "short" and "long" links are referred to as intermediate links. Links of this length represent a transition between the two types of link behavior. The detailing of intermediate links should ensure that the controlling inelastic energy dissipating mechanisms can be fully developed.

The ductility of an EBF is controlled by the inelastic rotation of the link segment. This rotation is measured as the angle between the link and the beam outside the link (Figure C709-1). Tests [Hjelmstad and Popov, 1983] have shown that when controlled by shear distortion, properly detailed short links with inelastic rotations of 0.10 radians performed adequately.

The link beam inelastic rotation limit for short links was set at 0.06 radians. This limit reflects some conservatism as well as uncertainty inherent in the $3(R_w/8)$ deflection calculations. At this level of short link rotation, limited test data have indicated that floor damage is small [Rickles and Popov, 1987].

Long links with lengths greater than 3.0 M/V have also been tested [Engelhardt and Popov, 1989]. The inelastic rotation limit was conservatively set at 0.015 radians. This limit is intended to preclude premature flange failure resulting from large bending strains at the link ends.

The inelastic rotations are evaluated at the braced end of the link beam. The angle measured is located between the extension of the line passing through the center of each end of the link beam and the line passing through the center of each end of the brace beam portion. The displacement of the beam end center points may be obtained geometrically by multiplying the drift angle obtained from the prescribed design force analysis by $3(R_w/8)$ and considering the elements of the EBF as rigid without flexural or axial deformation. This is, of course, slightly conservative since the elastic curvature of the beam segments between hinges, and the brace deformations do contribute a minor amount of the required deformation (Figure C709-1). It should be noted that the calculation of the rotation by multiplying the elastic deflections of the link beam by $3(R_w/8)$ would be less conservative, since these deflections include elastic effects, such as the axial deformation of the braces, which would not increase proportionally after the link begins to yield.

C709.5 UBC §2211.10.5 Link Beam Web

The prohibition of link beam web doubler plates was required because tests [Becker, 1971; Popov, 1980] have shown that they are not fully effective. Performance of eccentrically braced frames relies heavily on the predictability of the strength and strain features of the link beam. It is not advisable to complicate the behavior of the link beam by allowing holes within it.

The requirement that the prescribed seismic forces determined in UBC Chapter 16 shall not exceed 80 percent of the shear strength of the link beam is imposed to convert to working stress design. It establishes the same threshold for commencement of inelastic deformation as used for the other structural systems.

C709.6 UBC §2211.10.6 Brace-Beam Connection

The connection is required to develop the compressive strength level of the brace member. The prohibition of the extension of the brace-beam connection into the link beam is intended to prevent physical attachments that might alter the strength and deflection characteristics of the link beam or would be damaged by the large local rotations that can occur. It is not intended to prevent the center line intersection of brace and link beam from intersecting within the link. This intersection location is to be encouraged, for it results in a reduction in the moment that is required to be resisted by the brace and the beam beyond the link.

C709.7 UBC §2211.10.7 Link Beam Stiffeners

Web stiffeners have been found by cyclic testing to be necessary at the end of link beams. These stiffeners should be designed similar to column-web stiffeners, which transmit forces from the brace to the link beam. For links that may have significant bending yielding (length between 1.6 M_s/V_s and 3.0 M_s/V_s), an additional stiffener at a distance b_f from each end of the link is required to preclude the possibility of flange buckling (Figure C709-2).

C709.8 UBC §2211.10.8 Intermediate Stiffeners

Intermediate stiffeners should be provided for shear links to prevent premature buckling of the web. This will ensure development of the shear strength of the web and additionally reduce the cyclic deterioration of the web that can occur from repetition of buckling deformations (Figure C709-2).

C709.9 UBC §2211.10.9 Web Stiffener Spacing

The web stiffener spacing requirements are the result of numerous experimental and analytical studies on shear link subassemblages. The final recommendations presented were first described in Malley and Popov [1984].

C709.10 UBC §2211.10.10 Web Stiffener Location

Tests [Popov and Tsai, 1987] have shown that for member sizes below 18 inches in depth, single-sided full-depth web stiffeners provide adequate web stability for shear link elements. The 24-inch limitation was added since there have been no tests on members deeper than 18 inches. The minimum thickness requirement is an empirical criterion that should provide adequate stiffener performance. The minimum width requirement is included to provide the stiffener with enough rigidity to restrain web buckling from multiple panels.

C709.11 UBC §2211.10.11 Stiffener Welds

The weld between the web and stiffener should be adequate to develop the strength of the stiffener. The weld between the stiffener and the link flange is provided to increase the stiffener's rigidity and restrain any local flange buckling.

C709.12 UBC §2211.10.12 Link Beam-Column Connection

Testing indicates that long link postyield behavior is dominated by large, nonuniformly distributed, inelastic bending strains concentrated at the link end, which can result in premature fracture at link flange-column welds at low inelastic rotations. In the absence of further study, and until such time as specific recommendations can be developed that will ensure that the link-column connection is adequate to develop the required inelastic rotation of the link, it is recommended that placing long links adjacent to columns be avoided [AISC, 1992]. Short link beams adjacent to the column place large shear and moment demands on the beam-column connection. Testing has shown the need to make these connections as strong in shear as possible by providing full penetration groove welding of the link beam flanges, as well as by providing welded connections of the web for shear transfer. Bolted web connections are not felt to be acceptable under these conditions, since tests have shown that the large shear demand placed on these connections can lead to bolt slip and premature failure of link flange welds [Popov and Tsai, 1987].

The 1994 Northridge earthquake caused damage to steel moment resisting connections, which generated testing research at the University of Texas at Austin [Engelhardt and Sabol, 1994]. These tests included fully welded web connections to large girder sections (W 36 x 150), which are similar to the detail required by this section for short link-column connections. The test results did not reproduce the more satisfactory performance demonstrated by similar previous tests at Berkeley [Popov and Stephen, 1970] on smaller sections. It is important to note that the recent tests [Engelhardt and Sabol, 1994] did not induce shear yielding, which limits the inelastic stress and strain required of the flange-column welds that failed. The designer is cautioned to carefully consider the use of such links and the connection design, especially for large sizes, until further research is available.

C709.13 UBC §2211.10.13 Brace and Beam Strength

The ratio of axial compressive and reduced bending strength of the brace and the beam segment outside the link to the initial yield strength of the link beam was established at 1.5. The shear strength of the link beam is generally greater than the assumed strength based on 0.55 F_ydt, and can be more than 0.90 F_ydt with close stiffener spacing. The 1.5 ratio is considered the minimum that will maintain the integrity of the brace and the beam outside the link, and should also be applied to brace moments as well when these are significant. EBF geometry's that result in small angles between the beam and brace (less than 40 degrees) can cause very high axial forces in the beam outside the link. When combined with the moment transferred from the link portion of the beam, the high axial forces may make the 1.5 overstrength factor on link capacity difficult to attain without large braces and/or cover plates on the beams. Configurations with steep braces and short link beams can generally be designed to avoid these difficulties, provided the brace-beam connection resists a portion of the link end moment. Conservatism is recommended in sizing the braces, particularly for tube sections with thin wall thickness relative to their width that have limited cyclic postbuckling capacity.

C709.14 UBC §2211.10.14 Column Strength

The intent of this provision is to ensure that the initial inelastic behavior will occur in the link beams rather than in the columns. The requirement may be satisfied by showing that the columns have the strength to resist gravity load combined with 1.25 times the flexural and/or axial loads induced in the columns when the controlling strength of the link beams is developed, as defined in Section C709.3. It must be recognized that the requirement does not protect the bottom base from hinging. When a fixed base detail is used, such hinging may occur if the drift of the frame becomes large enough. It is recommended that base level columns be compact sections and that the base connection be detailed to recognize and accommodate potential hinging without failure of the column or of the brace-column connection.

C709.15 UBC §2211.10.15 Roof Link Beam

Link beams are not required in roof beams for EBFs over five stories in height because inelastic analyses have shown only limited demand on these links in taller buildings.

C709.16 UBC §2211.10.16 Concentric Brace in Combination

One exception is provided to the requirement that all braces be connected to at least one link. This allows the use of concentric bracing in the first story of EBFs over five stories if this story has an elastic capacity 50 percent greater than the yield capacity of the frame above. This was included in order to allow for a continuous framing configuration for EBFs where the link beam is at the bottom of the brace

(Figure C706-2) or to provide the opportunity for the designer to control the story drift of tall first story buildings.

C709.17 UBC §2211.10.17 Axial Forces

The axial forces to be considered should be those required to cause yielding of the link beam. It is further recommended that collectors connecting directly to the ends of EBFs be designed and connected to provide sufficient strength to deliver the forces corresponding to link beam yield and dissipate them into the diaphragm.

C709.18 UBC §2211.10.18 Beam Flanges

Key elements necessary for the development of the full strength and rotation capacity of shear links are proper stiffening and lateral bracing. Lateral bracing must be provided at the link ends and is critical to the stability of both the link and the brace. The function of the lateral bracing is not only to restrain lateral torsional buckling of the beam, but also to hold the brace end within the plane of the frame; any out-of-plane eccentricity of the brace end will produce twisting of the beam and the link whenever the brace is in compression. A composite deck by itself cannot be counted on to provide adequate lateral support for the link ends, and therefore, transverse beams are the preferred lateral bracing system. If diagonal braces are provided that extend from the bottom flange of a link to the top of a floor beam that is parallel to the link, consideration should be given to the kinematics of the link end moments relative to the parallel beam. In a study on long links, the out-of-plane forces on virtually all the test specimens were significantly larger, 4-6 times larger, than the previously recommended end bracing force of 1.5 percent of the beam flange strength. Various forms of buckling, such as lateral torsional buckling of the beam segments outside the link or that of the link itself, and severe web buckling within the link were among the contributing factors in development of large link end lateral forces. Resistance to these out-of-plane forces can be achieved by adequately stiffening the link end lateral support. Use of a minimum of 6 percent of the beam flange strength to support the link ends [Engelhardt and Popov, 1989] is recommended.

C709.19 UBC §2211.10.19 Beam-Column Connection

If the link beam is not located adjacent to the column, the requirements for connection of the brace at the beam-column intersection is the same as for concentrically braced frames. The connection must be able to resist beam rotation due to accidental eccentricities out of the plane of the frame from the brace. A torsional moment of 0.01 $F_y b_f t_f d$ is required to be resisted by the connection in order to prevent beam rotation under the large loads of the brace, without otherwise limiting the design of the connection as required by Section C709.12. Collector forces as described in C709.17 must be considered.

C710 UBC §2211.11 Light Framed Wall Systems

The criteria developed for stud wall systems apply to bearing wall structures framed with cold-formed studs and joists. While lateral forces are usually designed to be resisted by pairs of tension only diagonal bracing, additional redundancy and stiffness are typically provided by wall sheeting materials that are not included in the lateral force design. Where this redundancy is nonexistent, the engineer should provide remedial solutions.

These slender braces should remain essentially elastic during an earthquake to provide needed drift control. If braces are allowed to yield, the structural system becomes "loose" and drift control may be compromised.

Elsewhere in the UBC provisions, where there was a desire for a system or component to remain essentially elastic during a strong motion earthquake, the design lateral forces were increased by a $3(R_w/8)$ factor. The stud wall system was assigned an R_w factor of 4. This results in higher design lateral forces than previously required. It was the consensus of SEAOC that this reduced R_w factor of 4 by itself would result in an adequate design procedure. Consideration was given to the additional redundancy provided by the wall sheeting materials attached to the studs.

Where the wall sheeting material is attached on one side only, these Requirements state that the unsheeted stud flanges must be braced to prevent lateral torsional buckling.

While these Requirements recognize the benefits of the wall cladding redundancy, the connections of diagonal braces, top chord splices, and boundary and collector members were shown in past earthquakes to be the weak links of this system. These types of connection need to be protected, preferably by requiring elastic performance. The approach was to require these connections to either develop the strength of connected members or be designed for $3(R_w/8)$ times the prescribed seismic forces.

Tests of braced bay anchorage have shown that premature failure may be expected if the bottom track web resists uplift through bending resistance. Thus, these Requirements call for positive anchorage of the end stud brace or other means to avoid such conditions.

References

AISC, 1992. "Seismic provisions for Structural Steel Buildings," *Manual of Steel Construction, Load and Resistance Factor Design (Volume I)*, American Institute of Steel Construction, Chicago, Illinois.

AISC, 1994a. *Load and Resistance Factor Design Specification for Structural Steel Buildings*. American Institute of Steel Construction, Chicago, Illinois.

AISC, 1994b. Advisory Committee on SMRF Research, *Northridge Steel Update 1*, American Institute of Steel Construction, Chicago, Illinois.

AISC, 1989. "Specification for Structural Steel Buildings, Allowable Stress Design and Plastic Design," *Load and Resistance Factor Design Specification for Structural Steel Buildings*. American Institute of Steel Construction, Chicago, Illinois.

AISI, 1994. "Statistical Analysis of Tensile Data for Wide Flange Structural Shapes," American Iron and Steel Institute, Washington, D.C.

Anderson, J.C. and R.R. Linderman, 1991. *Steel Beam to Box Column Connections*, Report No. CE 91-03, Los Angeles: University of Southern California.

Aslani, F., and S.C. Goel, 1991. "Stitch Spacing and Local Buckling in Seismic-Resistant Double-Angle Braces," *ASCE Journal of Structural Engineering*, Volume 177, no. 8.

Astaneh, A., S.C. Goel, and R.D. Hanson, 1986. "Earthquake-Resistant Design of Double-Angle Bracing," *AISC Engineering Journal*, Fourth Quarter.

ATC-24, *Guidelines for Cyclic Seismic Testing of Components of Steel Structures,* Applied Technology Council, Redwood City, California.

Basha, H., and S.C. Goel, October 1994. "Seismic Resistant Truss Moment Frames with Ductile Vierendeel Segment," Research Report No. UMCEE 94-29, Department of Civil and Environmental Engineering, The University of Michigan, Ann Arbor, MI.

Becker, E. R., 1971. *Panel one Effect on the Strength of Rigid Steel Frames,* University of Southern California Structural Mechanics Laboratory, USCOE-001.

Bertero, V., J. Anderson, and H. Krawinkler, 1994. *Performance of Steel Building Structures During the Northridge Earthquake*, Report No. UCB/EERC-94/09, Berkeley: Earthquake Engineering Research Center.

Butler, L.J., and G.L. Kulak, 1971. "Strength of Fillet Welds as a Function of Direction of Load," *Welding Journal*, American Welding Society.

Engelhardt, M.D. and A. S. Husain, 1993. "Cyclic-Loading Performance of Welded Flange-Bolted Web Connections," *Journal of Structural Engineering,* American Society of Civil Engineers, New York.

Engelhardt, M.D. and Popov, E.P., 1989. "On Design of Eccentrically Braced Frames," *Earthquake Spectra*, Vol. 5, no. 3, Earthquake Engineering Research Institute, Oakland, California.

Engelhardt, M.D. and E.P Popov,1989. *Behavior of Long Links in Eccentric Braced Frames*, Report No. EERC-89/1. Earthquake Engineering Research Center, University of California, Berkeley, California.

FEMA, 1995. *Interim Guidelines: Evaluation, Repair, Modification and Design of Steel Moment Frames*. Report No. SAC-95-02. Sacramento, California.

Foutch, D. A., 1989. "Seismic Behavior of Eccentrically Braced Steel Building," ASCE *Journal of Structural Engineering*, Vol. 115, no. 8.

Goel, S. C., 1992a. "Cyclic Post Buckling Behavior of Steel Bracing Members," *Stability an Ductility of Steel Structures Under Cyclic Loading*, CRC Press.

Goel, S. C., 1992b. "Earthquake-Resistant Design of Ductile Braced Steel Structures," *Stability and Ductility of Steel Structures Under Cyclic Loading,* CRC Press.

Goel, S. C., and S. Lee, 1992. "A Fracture Criterion for Concrete Filled Tubular Bracing Members Under Cyclic Loading, " *Proceedings, ASCE Structures Congress*, San Antonio, TX, April 13-15.

Goel, S. C., and R.D. Hanson, 1987. "Behavior of Concentrically Braced Frames and Design of Bracing Members for Ductility," SEAOC Proceedings, Structural Engineers Association of California, Sacramento, California.

Goel, S. C., and A. Itani, June 1994a. "Seismic Behavior of Open Web Truss Moment Frames," ASCE *Journal of Structural Engineering*.

Goel, S. C., and A. Itani, June 1994a. "Seismic Behavior of Special Truss Moment Frames," ASCE *Journal of Structural Engineering*.

Hassan, O., and S. C. Goel, 1991. *Seismic Behavior and Design of Concentrically Braced Steel Structures*, Report No. UMCE 91-1, Department of Civil Engineering, The University of Michigan, Ann Arbor, Michigan.

Hjelmstad, K. D., and E.P. Popov, 1983. *Seismic Behavior of Active Beam Links in Eccentrically Braced Frames*, Earthquake Engineering Research Center, University of California, Berkeley, California.

Itani, A., and S. C. Goel, September 1991. "Earthquake Resistance of Open Web Steel Framing Systems," Research Report No. UMCE 91-21, Department of Civil Engineering, The University of Michigan, Ann Arbor, MI.

Jain, A., S. C. Goel, and R.D. Hanson, 1980. "Hysteretic Cycles of Axially Loaded Steel Members," ASCE, ST8.

Kasai, K. and E.P. Popov, 1984. "On Seismic Design of Eccentrically Braced Steel Frames," *Proceedings, 8th World Conference on Earthquake Engineering,* San Francisco, California., Vol. 5.

Kasai, K. and E.P. Popov, 1986. *A Study of Seismically Resistant Eccentrically Braced Frames,* Report No. EERC-86/01, Earthquake Engineering Research Center, University of California, Berkeley, California.

Kasai, K., and E.P. Popov, 1986. "General Behavior of WF Steel Shear Link Beams," *Journal of the Structural Division of ASCE*, Vol. 112, no. 2.

Kasai, Kuzuhiko, and E.P. Popov, 1986. "Cyclic Web Buckling Control for Shear Link beams," *Journal of Structural Engineering*, ASCE, Vol. 112, no. 3.

Krawinkler, H., V.V. Bertero, and E.P. Popov, 1971. *Inelastic Behavior of Steel Beam-to-Column Subassemblages,* Report No. EERC 71-7, Earthquake Engineering Research Center, University of California, Berkeley, California.

Krawinkler, Helmut, 1978. "Shear in Beam-Column Joints in Seismic Design of Steel Frames," *Engineering Journal,* AISC, Volume 15, no. 3.

Lee, S., and S.C. Goel, 1987. *Seismic Behavior of Hollow and Concrete Filled Square Tubular Bracing Members*, Report No. UMCE 87-11, Department of Civil Engineering, University of Michigan, Ann Arbor, Michigan.

Lee, H. S., and S.C. Goel, 1990. *Seismic Behavior of Steel Built-Up Box-shaped Bracing Members and Their Use in Strengthening RC Frames*, Report No. UMCE 90-7, Department of Civil Engineering, The University of Michigan, Ann Arbor, MI.

Malley, J. O. and E.P. Popov, 1983. *Design Considerations for Shear Links in Eccentrically Braced Frames*, Report No. EERC-83/23, Earthquake Engineering Research Center, University of California, Berkeley, California.

Malley, J.O., and E.P. Popov, 1984. "Shear Links in Eccentrically Braced Frames," *Journal of Structural Engineering*, ASCE, Vol. 110, no. 9.

Paulay, T., 1983. "Deterministic Seismic Design Procedures for Reinforced Concrete Buildings," Engineering Structures, Vol. 5, no. 1.

Popov E. P. and K. C. Tsai, 1987. "Performance of Large Seismic Steel Moment Connections under Cyclic Loads," *Proceedings 56th SEAOC Convention*, Structural Engineers Association of California, Sacramento, California.

Popov, E. P. and V. Bertero, 1973. "Cyclic Testing of Steel Beams and Connections," *Journal of the Structural Division,* American Society of Civil Engineers, New York.

Popov, E. P., "Panel Zone Flexibility in Seismic Moment Joints," 1987. *J. Construct. Steel Research*, Vol. 8, Elsevier Applied Science Publishers Ltd., England.

Popov, E. P., 1988. "On California Structural Steel Seismic Design," *Earthquake Spectra*, EERI, Vol. 2, no. 4, Earthquake Engineering Research Institute, Oakland, California.

Popov, E. P., 1980. "Seismic Behavior of Structural Assemblages," Journal of the Structural Division, ASCE, Vol. 106, no. ST7.

Popov, E. P., and R.G Black, 1981. "Steel Struts Under Severe Cyclic Loadings," ASCE, ST9.

Popov, E. P., and R.B. Pinkney, 1969. "Cyclic Yield Reversal in Steel Building Connections," *Journal of the Structural Division*, ASCE, Vol. 95, no. ST3.

Popov, E. P., and R.M. Stephen, 1970. *Cyclic Testing of Full Size Steel Connections,* Report No. EERC-70-3, Earthquake Engineering Research Center, University of California, Berkeley, California.

Popov, E. P., V.V. Bertero, and Chandramouli, S., 1975. "Hysteretic Behavior of Steel Columns," Report No. EERC 75-11, Earthquake Engineering Research Center, University of California, Berkeley, California.

Popov, E. P., M.D. Engelhardt, and J.M. Ricles, 1989. "Eccentrically Brace Frames: U. S. Practice," *Engineering Journal*, American Institute of Steel Construction, Chicago, IL., Vol. 26, no. 2.

Popov, E. P., Kasai, K., and M.D. Engelhardt., 1987. "Advances in the Design of Eccentrically Braced Frames," *Earthquake Spectra,* Vol. 3, no 1, Earthquake Engineering Research Institute, Oakland, California.

Popov, E.P., N.R. Amin, J.C. Louie, and R.M. Stephen, 1986. "Cyclic Behavior of Large Beam-Column Assemblies," Engineering Journal, AISC Volume 23, no. 1.

Rickles, J. M. and E.P. Popov, 1987. "Dynamic Analysis of Seismically Resistant Eccentrically Braced Frames," Report No. EERC-87/07, Earthquake Engineering Research Center, University of California, Berkeley, California.

Roeder, C. W. and E.P. Popov, 1977. "Inelastic Behavior of Eccentrically Braced Steel Frames Under Cyclic Loadings," Report No. EERC-77/18, Earthquake Engineering Research Center, University of California, Berkeley, California.

Roeder, C. W., 1987. "Inelastic Dynamic Analysis of Two Eight Story Moment Frames," Report to Structural Engineers Association of Washington.

Roeder, C. W., and E.P. Popov, 1978. "Cyclic Shear Yielding of Wide Flange Beams," Journal of the Engineering Mechanics Division, ASCE, Vol. 104, no. EM4.

SAC, 1994. *International Workshop on Steel Seismic Issues*, Structural Engineers Association of California, Applied Technology Council, and California Universities for Research in Earthquake Engineering (SAC Joint Venture). Sacramento, California.

SAC, 1994. *Steel Moment Frame Advisory No. 1,* SAC Joint Venture, Sacramento, California.

SAC, 1994. Internal Working Document, *Steel Moment Frame Connection Advisory No. 2*, SAC Joint Venture, Sacramento, California.

SAC, 1995. *Steel Moment Frame Advisory No. 3*, SAC Joint Venture, Sacramento, California.

SEAOC, 1994. *Ductile Steel Frame Beam-Column Joints,* A Discussion of Preliminary Observations, Conclusions, and Recommendations, Seismology Committee, Structural Engineers Association of California, Sacramento, California.

Slutter, R., 1981. "Tests of Panel Zone Behavior in Beam-Column Connections," Lehigh 200.81.403.1.

SSPC, 1994. *Statistical Analysis of Tensile Data for Wide-Flange Structural Shapes.* Structural Shape Producers Council.

Tang, X., and S.C. Goel, 1987. "Seismic Analysis and Design Considerations of Braced Steel Structures," Report No. UMCE 87-4, Department of Civil Engineering, The University of Michigan, Ann Arbor, Michigan.

Tang, X., and S.C. Goel, 1989. "Brace Fractures and Analysis of Phase 1 Structure," Journal of Structural Engineering, ASCE, Vol. 115, no. 8.

Tsai, K-C, and E.P. Popov, 1988. "Steel Beam-Column Joints in Seismic Moment Resisting Frames," Report No. EERC-88/19, Earthquake Engineering Research Center, University of California, Berkeley, California.

Wallace, B. J. and H. Krawinkler, 1985. "Small-Scale Model Experimentation on Steel Assemblies," *U.S.-Japan Research Program,* John A. Blume Earthquake Engineering Center Report No. 75, Department of Civil Engineering, Stanford University, California.

Whittaker, A. S., U.M. Uang, and V.V. Bertero, 1987. *Earthquake Simulation Tests and Associated Studies of a 0.3-Scale Model of a Six-Story Eccentrically Braced Steel Structure*, Report No. EERC-87/02, Earthquake Engineering Research Center, University of California, Berkeley, California.

Xu, P. and S.C. Goel, 1990. "Behavior of Double Channel Bracing Members Under Large Cyclic Deformations," Report No. UMCE 90-1, Department of Civil Engineering, The University of Michigan, Ann Arbor, MI.

Zayas, V. A., P. Shum, B. Shing, S.A. Mahin, and E.P. Popov, 1981. *Inelastic Offshore Platforms for Seismic Loading*, EERC 81/04, Earthquake Engineering Research Center, University of California, Berkeley, California.

Table C703-1. *Summary Statistics From AISC Yield Strengths Survey [AISI, 1994]*

Statistic	A36	Dual	A572
Number of Data	36,570	7,824	13,536
Yield Point (ksi)			
Coefficient of variation	0.100	0.066	0.089
Standard deviation (SD)	4.9	3.7	5.1
Maximum value	72.4	71.1	79.5
Mean+2SD	59.1	62.5	67.9
Mean	49.2	55.2	57.6
Median	48.9	54.6	57.0
Mean-2SD	39.4	47.8	47.4
Minimum value	36.0	50.0	50.0
Tensile Strength (ksi)			
Coefficient of variation	0.67	0.045	0.082
Standard deviation (SD)	4.6	3.3	6.2
Maximum value	88.5	80.0	104.0
Mean+2SD	77.6	79.8	88.0
Mean	68.5	73.2	75.6
Median	68.3	73.2	75.0
Mean-2SD	59.3	66.6	63.2
Minimum value	58.0	65.0	65.0
Yield/Tensile Ratio			
Coefficient of variation	0.080	0.050	0.063
Standard deviation (SD)	0.057	0.038	0.048
Maximum value	0.925	0.922	0.954
Mean+2SD	0.835	0.829	0.860
Mean	0.720	0.754	0.763
Median	0.721	0.749	0.758
Mean-2SD	0.605	0.678	0.666
Minimum value	0.512	0.653	0.618

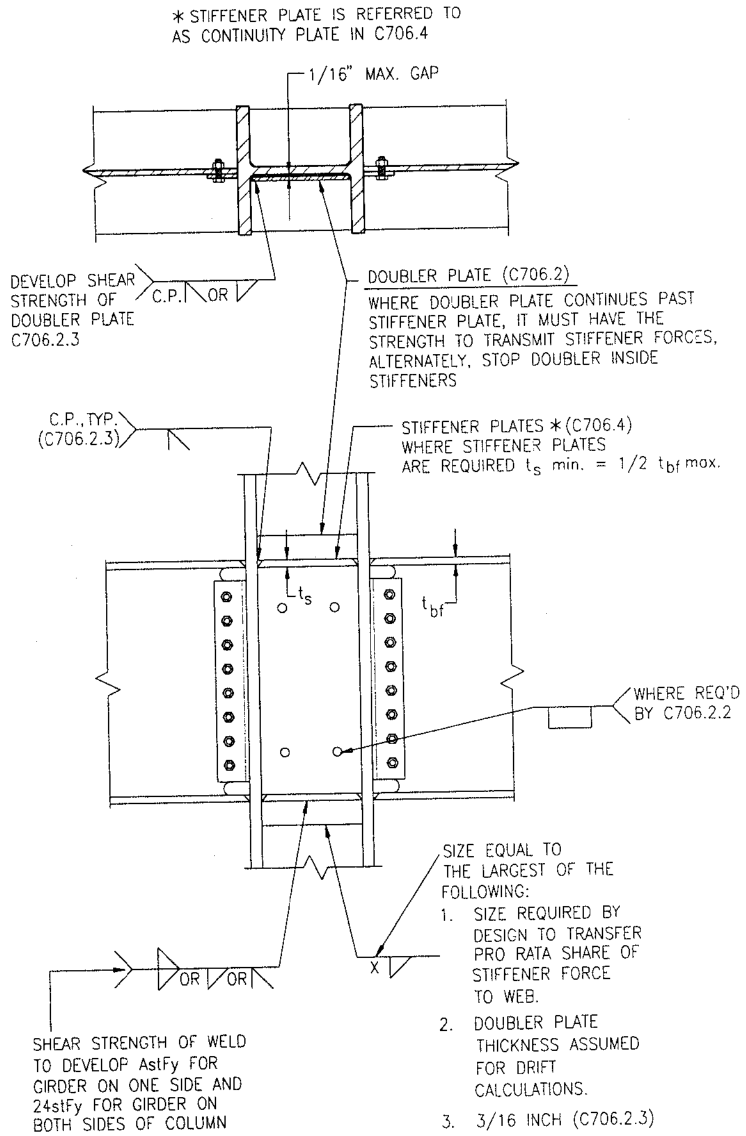

* STIFFENER PLATE IS REFERRED TO AS CONTINUITY PLATE IN C706.4
1/16" MAX. GAP
DEVELOP SHEAR STRENGTH OF DOUBLER PLATE C706.2.3
C.P.
OR
DOUBLER PLATE (C706.2)
WHERE DOUBLER PLATE CONTINUES PAST STIFFENER PLATE, IT MUST HAVE THE STRENGTH TO TRANSMIT STIFFENER FORCES, ALTERNATELY, STOP DOUBLER INSIDE STIFFENERS
C.P., TYP. (C706.2.3)
STIFFENER PLATES * (C706.4) WHERE STIFFENER PLATES ARE REQUIRED t_s min. = 1/2 t_{bf} max.
t_s
t_{bf}
WHERE REQ'D BY C706.2.2
SIZE EQUAL TO THE LARGEST OF THE FOLLOWING:
1. SIZE REQUIRED BY DESIGN TO TRANSFER PRO RATA SHARE OF STIFFENER FORCE TO WEB.
2. DOUBLER PLATE THICKNESS ASSUMED FOR DRIFT CALCULATIONS.
3. 3/16 INCH (C706.2.3)
X
OR
OR
SHEAR STRENGTH OF WELD TO DEVELOP AstFy FOR GIRDER ON ONE SIDE AND 2AstFy FOR GIRDER ON BOTH SIDES OF COLUMN

Figure C706-1. *Stiffener and doubler plates*

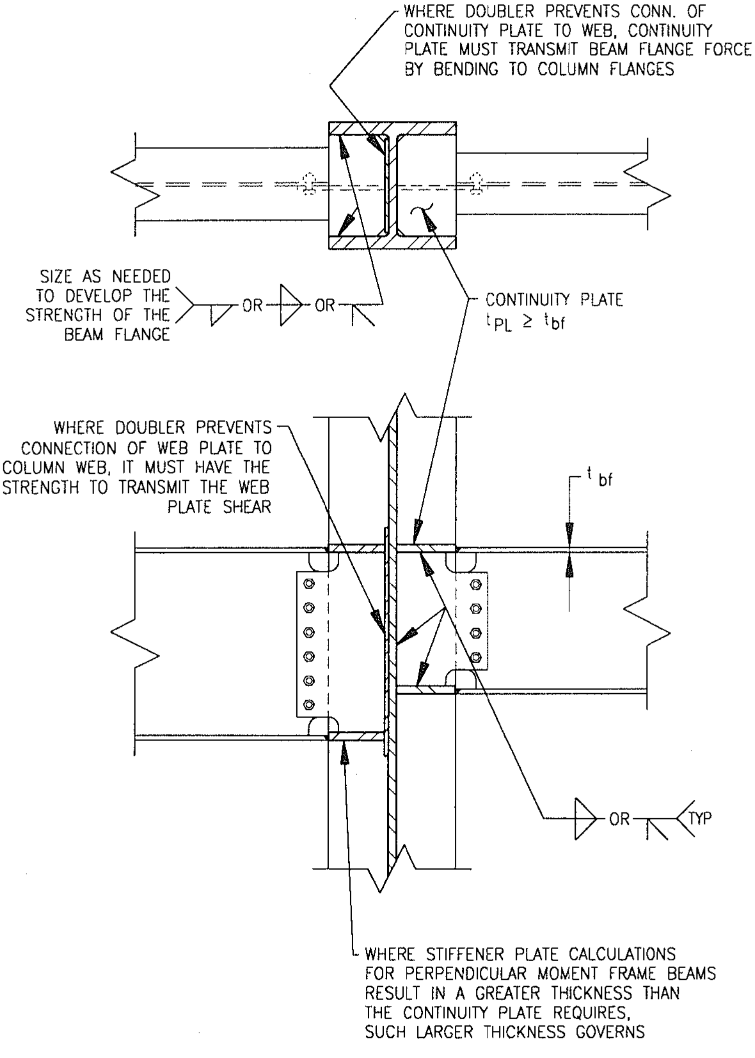

WHERE DOUBLER PREVENTS CONN. OF CONTINUITY PLATE TO WEB, CONTINUITY PLATE MUST TRANSMIT BEAM FLANGE FORCE BY BENDING TO COLUMN FLANGES
SIZE AS NEEDED TO DEVELOP THE STRENGTH OF THE BEAM FLANGE
OR
OR
CONTINUITY PLATE
$t_{PL} \geq t_{bf}$
WHERE DOUBLER PREVENTS CONNECTION OF WEB PLATE TO COLUMN WEB, IT MUST HAVE THE STRENGTH TO TRANSMIT THE WEB PLATE SHEAR
t_{bf}
OR
TYP
WHERE STIFFENER PLATE CALCULATIONS FOR PERPENDICULAR MOMENT FRAME BEAMS RESULT IN A GREATER THICKNESS THAN THE CONTINUITY PLATE REQUIRES, SUCH LARGER THICKNESS GOVERNS

Figure C706-2. *Continuity plate detail*

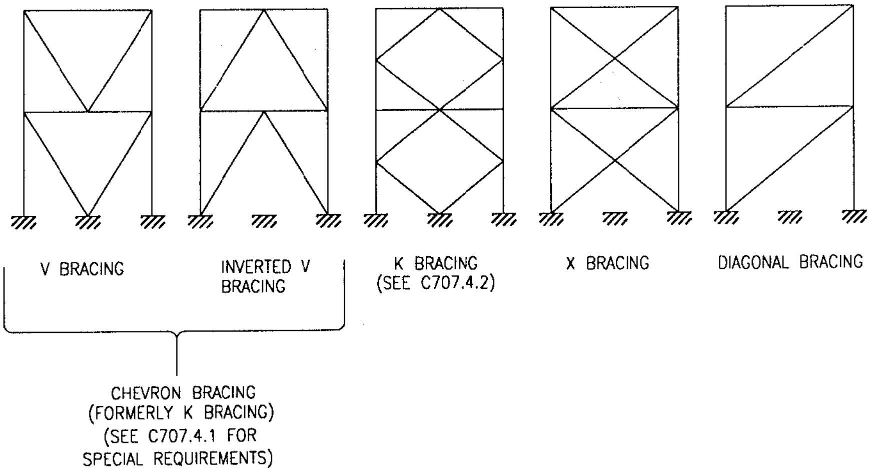

V BRACING
INVERTED V BRACING
K BRACING (SEE C707.4.2)
X BRACING
DIAGONAL BRACING
CHEVRON BRACING (FORMERLY K BRACING) (SEE C707.4.1 FOR SPECIAL REQUIREMENTS)

Figure C707-1. *Examples of concentric bracing configurations*

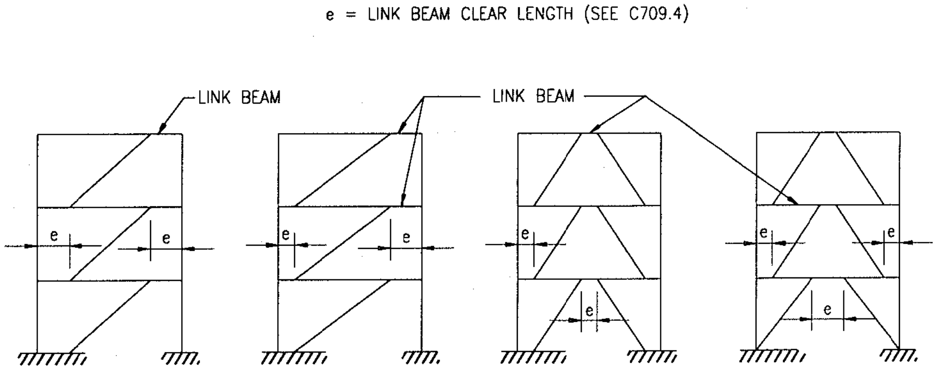


Figure C707-2. *Examples of eccentric bracing configurations*

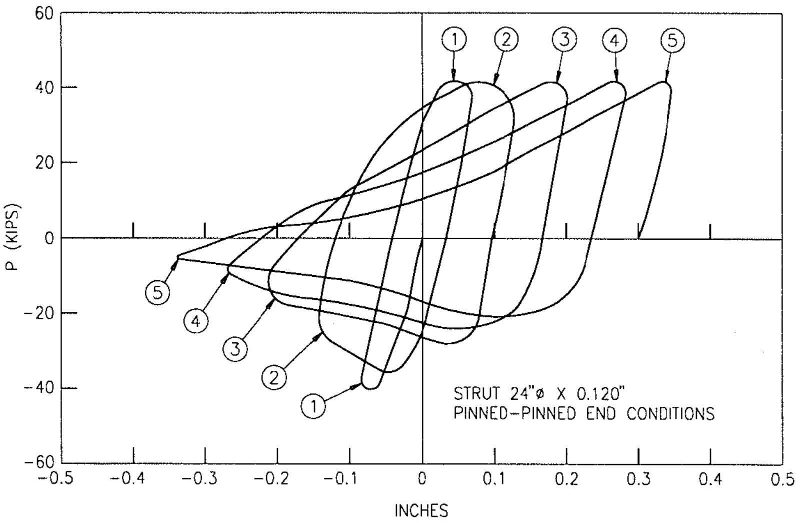


Figure C707-3. Cyclic response of an axially loaded element [Zayas et al.]

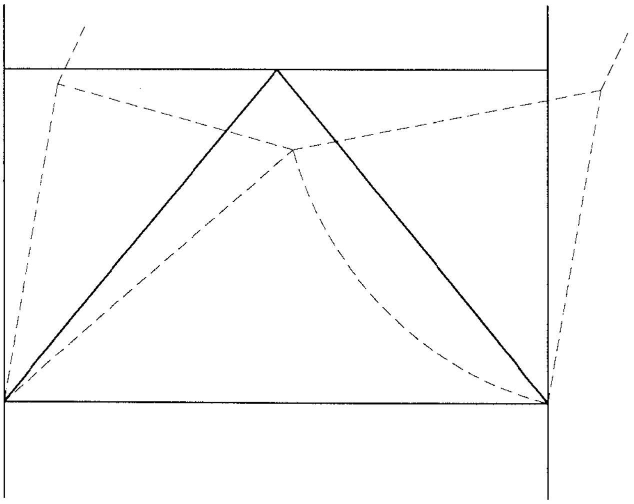


Figure C707-4. Chevron bracing in postbuckling stage

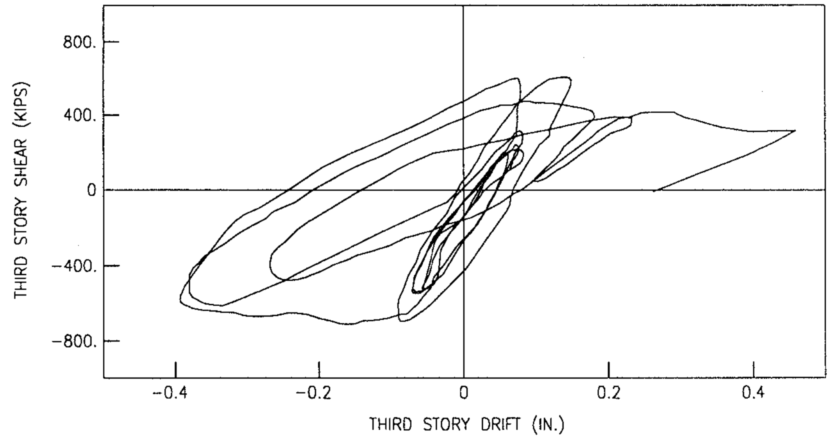


Figure C707-5. *Story shear/story drift diagram for a frame structure with chevron bracing [Wallace and Krawinkler, 1995]*

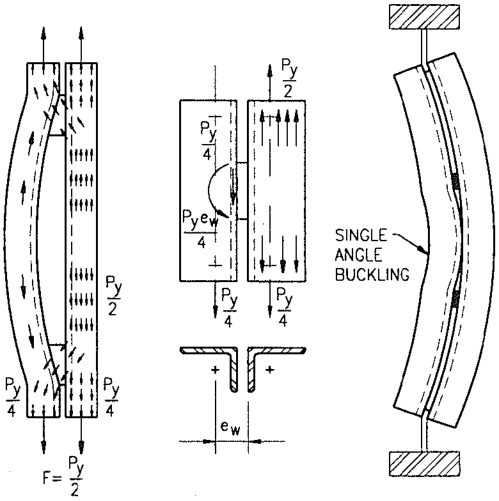


Figure C708-1. *Design forces for stitches in out-of-plane buckling bracings*

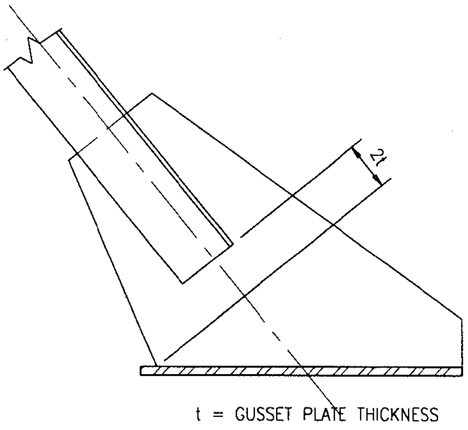


Figure C708-2. *Plastic hinge and free length of gusset plate*

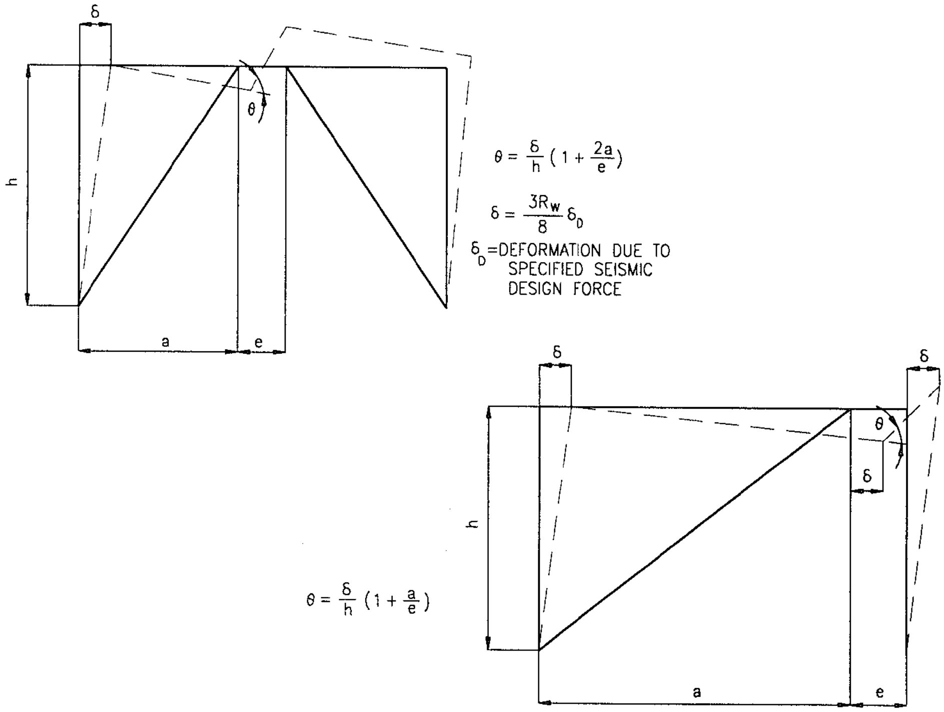

δ
θ
h
a
e
$\theta = \frac{\delta}{h}(1 + \frac{2a}{e})$
$\delta = \frac{3R_w}{8}\delta_D$
δ_D =DEFORMATION DUE TO SPECIFIED SEISMIC DESIGN FORCE
δ
δ
θ
δ
h
$\theta = \frac{\delta}{h}(1 + \frac{a}{e})$
a
e

Figure C709-1. *Eccentrically braced frames; examples of deformed geometry showing relationships*

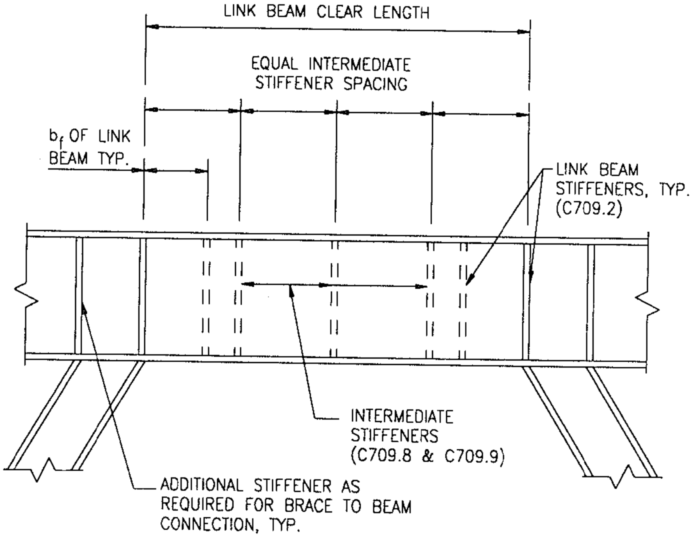

LINK BEAM CLEAR LENGTH
EQUAL INTERMEDIATE STIFFENER SPACING
b_f OF LINK BEAM TYP.
LINK BEAM STIFFENERS, TYP. (C709.2)
INTERMEDIATE STIFFENERS (C709.8 & C709.9)
ADDITIONAL STIFFENER AS REQUIRED FOR BRACE TO BEAM CONNECTION, TYP.

Figure 709-2. *Link beam and intermediate stiffeners*

CHAPTER 8
Wood

C801 UBC §2301 General Requirements

SEAOC adopts the provisions of the 1994 UBC, with the modifications and additions specified in Section 802.

The wood construction provisions of the 1994 UBC are considered to be "state-of-the-art" for lateral force design. Permissible methods of construction, diaphragm ratios, diaphragm shears, stability criteria, and established connector values are listed in the code.

It is for this reason that the 1994 UBC is referenced in the first paragraph of Chapter 8. Any provisions recommended here assume stress levels, material grades, and inspection requirements at least equal to those required by the UBC. It should be noted that these Recommendations are based on the 1994 UBC and were completed before the adoption of the 1994 UBC.

There has been no attempt to address all of the construction materials in UBC Chapter 23, Wood.

The recommendations for wood construction appear in this chapter and Section 108.

C802 Recommended Modifications to 1994 UBC

C802.1 Safe Lateral Strength

C802.1.1 *Add new subsection:* ***UBC §2311.3.1.1 Toenails and Slant Nailing.*** This Provision limiting the transfer of lateral forces by the use of toenails and slant nails was added to limit the failure mechanism caused by the shrinking of blocking or the close spacing of nails that provide a weakened plane for splitting, as shown in Figure C802-5. The value of 150 pounds per lineal foot was selected based on the spacing of 8-penny nails at 6 inches on center and using the 1991 National Design Specifications for nail values. A slant nail factor of 1.0 was used because the orientation where the nail is driven perpendicular to grain for both members is slant nailed. While toenail/slant nail orientation value is recognized, positive shear transfer connections by use of face nailing to perpendicular blocking or framing anchors, as indicated in Figures C802-6 and C802-7, is preferable.

C802.1.2 *Add new subsection:* ***UBC §2314.3.1.2 Diaphragms.*** Due to observed results of the Northridge earthquake (see engineering implications of the Northridge earthquake, Appendix A), the vertical diaphragm ratios have been changed to limit horizontal displacement or drift of wood-framed buildings. The height-to-width ratios indicated in Figures C802-8 and C802-9 are anticipated for the calculation of height-to-width ratios.

C802.2 UBC §2314.1 Wood Shear Walls and Diaphragms in Seismic Zones 3 and 4

General. A design methodology has evolved over the years where the diaphragm is considered to transmit only shear and where tension or compression members are provided to resist moments (chord forces) or to distribute diaphragm shears (diaphragm struts or cross-ties). Even though this methodology is arbitrary, it can be used safely and with consistency. Where a diaphragm is interrupted by an opening, the diaphragm shears at the opening and immediately beyond the opening change abruptly. Ties and struts must be provided at these locations to distribute this change in force. The requirement for full width ties to distribute differential diaphragm shears may be relaxed for the case where small openings occur in large diaphragms.

UBC §2316.2 restricts the use of shear walls that support horizontal forces contributed by masonry and concrete to buildings one story in height. Recognizing the need for wood shear walls to support two-story walls rather than one, an exception is added where plywood sheathed walls can be used, with restrictions. In order to be able to support rigid walls laterally, a plywood shear wall must possess rigidity comparable to the supported element. To achieve this, a story height limit is established, a lower allowable shear is recommended, lateral deflection limits are imposed, and $^{15}/_{32}$ inches minimum plywood thickness is recommended. Conditions leading to torsional stresses due to rotation of the diaphragm are eliminated, and a planar continuity in shear transfer to eliminate secondary stresses was adopted.

C802.2.1–C802.2.3 No Commentary provided.

C802.2.4 ***UBC §2314.5.6 Sill Plates and Framing Members.*** The Blue Book recommendations for minimum size wood sills and framing of shear walls is in response to the 1996 UBC Supplement, which has set a 350 plf trigger for all framing in all seismic zones to require 3x framing. This is also driven by Los Angeles Basin-ICBO requirements to maintain 1/2" panel edge distance for "highly stressed" shear walls.

Both of these requirements were in response to damage observed in the Northridge Earthquake without test data to justify the 350 plf as a trigger to define "highly stressed" shear walls. In the 1996 UBC Supplement, the requirement for 3x's will appear as a footnote to the subject tables. Because current footnote 2 provides requirements where panels are applied on both faces of the wall, this trigger is interpreted to mean both sides of the wall.

The American Plywood Association (APA) will be retesting plywood shear walls in late 1996 under cyclic loading to determine the load limit trigger for 3x members. Therefore, this version of the Blue Book defers to the verified load trigger of codes under development beyond its printing deadline.

C802.3 Conventional Light Frame Construction

C802.3.1 ***UBC §2326.1 General.*** The deletion of diagonal let-in bracing reflects the general practice of the engineering community and building departments within California. Also, deletion of the use of stucco/Portland Cement plaster for use in bracing cripple walls is compatible with Northridge earthquake observations, which demonstrated that Portland Cement plaster does not provide adequate resistance for cripple stud walls for a typical Seismic Zone 4 structure. Further consideration of the recommendation for restricting the use of Portland Cement plaster will be made after cyclic testing of such panels is completed.

C802.3.2 ***UBC §2326.11.5 Cripple Walls.*** The addition of Table 802-3 (see Requirements) is to differentiate between bracing required at the cripple wall level, where normally only exterior walls are provided, compared to bracing prescribed for multi-story levels where interior partitions can share the lateral load demands. The use of 3/8-inch-thick wood structural panel was selected as the most economical bracing material, with 8-penny common nails specified to produce the required capacity of the material. The question may arise whether the use of sinker nails will affect the capacity. If that is a concern, the minimum percentage of well-braced length could be increased accordingly, or 15/32-inch-thick panels with 10-penny nails could be substituted.

C802.3.3 ***UBC §2314.5.3 Wood Structural Panels.*** Providing exterior glue, minimum plywood sheet size, and blocking will improve the resistance of a shear wall subjected to in-plane loading. Providing large sheets will decrease the number of joints and therefore reduce the likelihood of secondary stresses around joints. The greater the number of joints, as in the case of smaller plywood sheets, the greater the exposure to failure due to the weak links generated by jointing; moreover, use of small sheets will cause the shear wall to be less stiff due to nail slippage.

C802.3.4 ***UBC §2314.5.4 Heavy Wood Panels.*** This Provision covers specifications for heavy deck materials such as plywood, diagonal, or straight decking used as diaphragms. Although used in common practice, the allowable shear loads of these materials or their application details specified in this Provision are not covered in the UBC.

C802.3.5 ***UBC §2314.5.6 Sill Plates and Framing Members.*** During recent earthquakes, it was noted that shear walls with high assigned lateral loads suffered major damage or complete failure due to inadequate nail edge distance. Insufficient nail edge distance was attributed to the use of 2-inch (51 mm) nominal framing members at adjoining panel edges and sill plates. In most cases, the plywood edges were sheared off at the nail holes due to lack of adequate edge distance. The addition was made to clarify the location of the 3x members and the need for single 3x members, rather than allowing two 2x members, which could result in splitting problems.

Additional Commentary

The following Commentary discusses selected sections of the 1994 UBC. Sections that do not reference a UBC section or paragraph present discussion of specific topics and do not refer to any specific UBC requirement.

C803 UBC §2302 Definitions

Because of their importance, the definitions of subdiaphragm, blocked diaphragm, and rotation found in UBC §2302 are discussed below.

Subdiaphragm. This concept originated as a device for transferring anchorage forces from masonry walls or other mass concentrations to wood diaphragms. See Figure C802-1. In Figure C802-1, the heavy lateral forces contributed by the masonry wall are transmitted into the subdiaphragm by means of anchor ties. The span or length of the subdiaphragm in the figure is between the end shear wall and the center diaphragm strut. The anchor ties must run the full depth of the subdiaphragm and be connected to it to develop the design tie force. The subdiaphragm must be designed as if it were going to act by itself in transferring anchor tie forces to the shear wall and the diaphragm strut.

The subdiaphragm depth generally determines the length of the anchor ties. However, the depth of the subdiaphragm should be sufficient to provide:

1. A length of anchor tie that can transfer the anchor forces from the wall into the diaphragm without the use of cross-grain tension at panel joints.
2. A depth necessary to transfer the subdiaphragm shear to the shear wall and/or continuous diaphragm drag strut.

Item 1 above deserves further discussion related to the direction of the framing. Wall forces can be effectively transferred into the diaphragm when the framing members are perpendicular to the wall. The usual length of the framing member and the type of connection used will provide adequate development length to transfer the wall forces to the diaphragm. However, where framing members and panelized sheathing joints are parallel to the wall, special detailing is required to obtain adequate development length to transfer the wall anchorage force into the subdiaphragm. If an anchorage force is not fully transferred into the depth of the subdiaphragm, the panelized sheathing can only transmit the force at the panel juncture through the nails in the framing member by cross-grain tension, which is not permitted at the ledger and should not be permitted at the panel joints. Since the behavior of the subdiaphragm is complex, it is suggested that the maximum span-to-width ratio be 2-1/2:1 (See C108.2.8.1).

Blocked Diaphragm. In panel diaphragm construction it is generally only necessary to provide blocking and nailing at the edges where required by stress. Occasionally, all edges must be blocked to decrease diaphragm deformation. All joints in panel shear walls (vertical panel diaphragms) must be blocked and nailed.

Rotation. Rotation is best explained by example: diaphragms in Figures C802-2 and C802-3 are not considered to have rotation because they can achieve equilibrium utilizing reactions parallel to the applied force. Figure C802-4 illustrates a building with rotation. The structure must rotate to develop a resisting couple required to maintain static equilibrium.

C804 UBC §1701.5.16 Special Inspection of Wood Construction

Referring to Requirements Section 202.1.2, it is common practice for building inspectors to observe plywood and its nailing; however, where stresses are high and nail spacing is critical, special inspection is recommended to ensure full conformance of the construction with recommended details. Furthermore, in addition to nailing, other components such as anchor bolts, tie-downs, diaphragm ties, drag struts, and nailing practices (which are of vital importance to the proper performance and stability of the shear wall) must also be inspected for compliance.

References

AITC, American Institute of Timber Construction, 1994, *Timber Construction Manual*, Fourth Edition, John Wiley & Sons, Inc., New York.

Hoyle, Robert J., Jr., and Frank E. Woeste, 1989, *Wood Technology in the Design of Structures*, Fifth Edition, Iowa State University Press, Ames, Iowa.

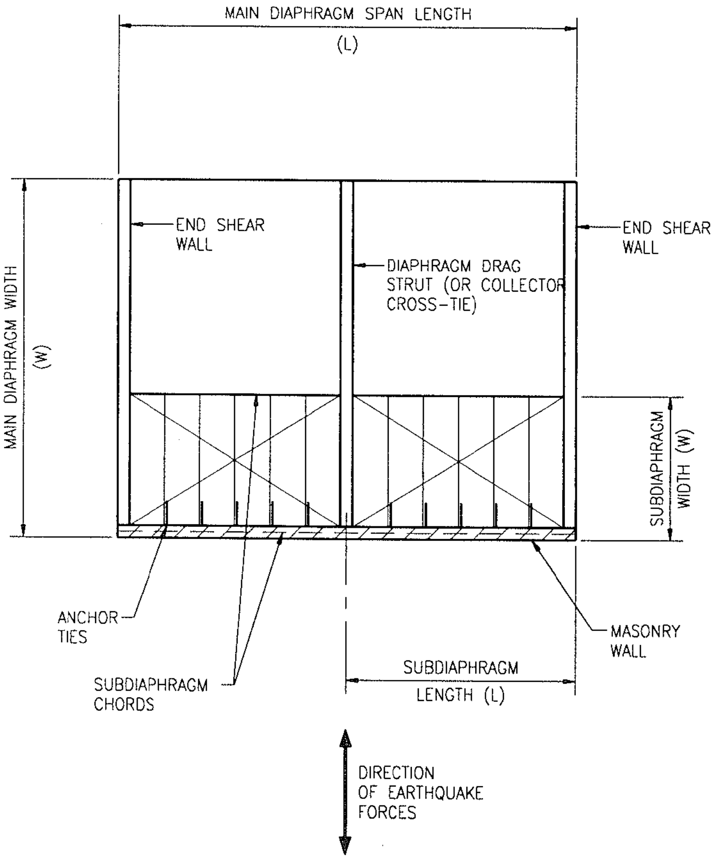


Figure C802-1. Subdiaphragm

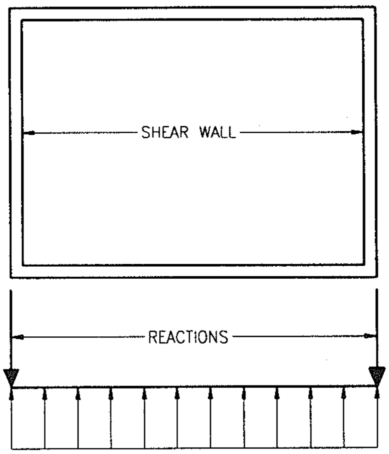


Figure C802-2. *Earthquake forces—regular diaphragm*

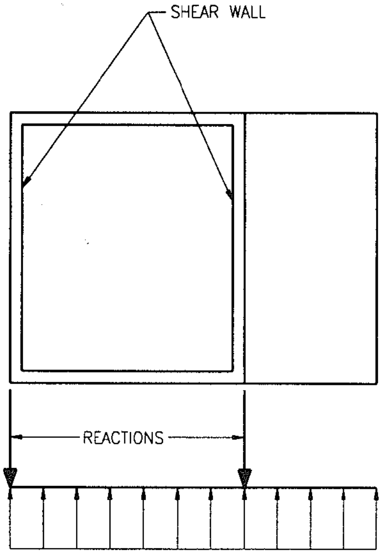


Figure C802-3. *Earthquake forces—cantilevered diaphragm*

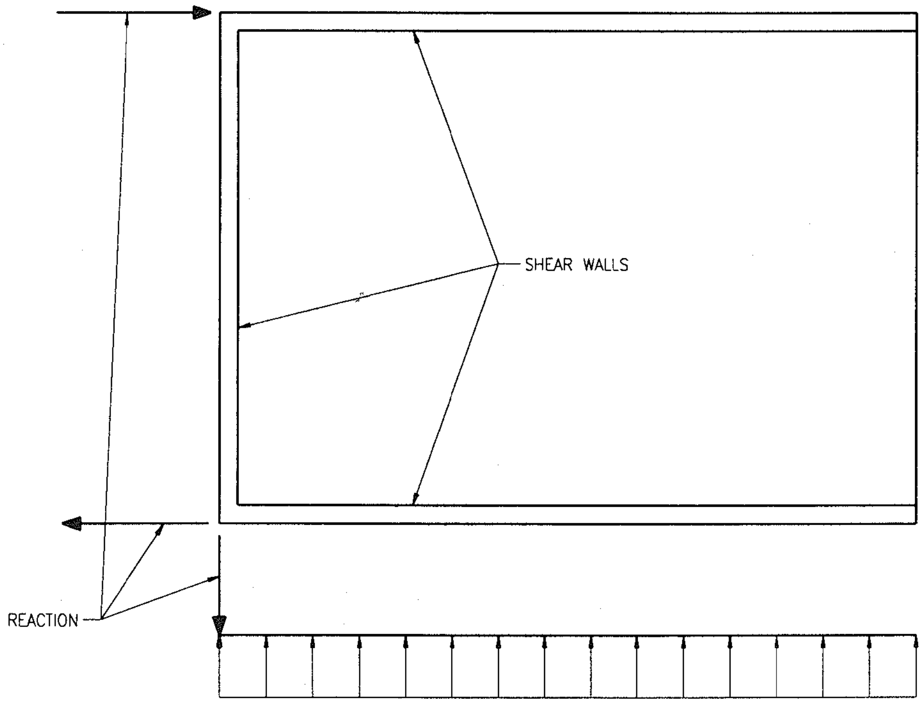


Figure C802-4. *Earthquake forces—building with rotation*

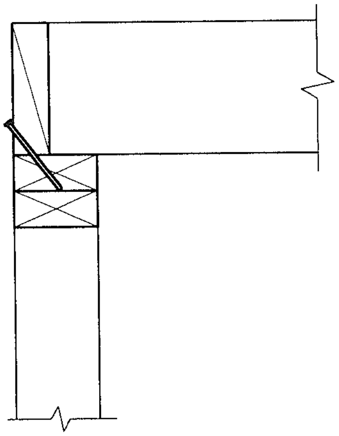


Figure C802-5. *Shear transfer utilizing slant/toenails*

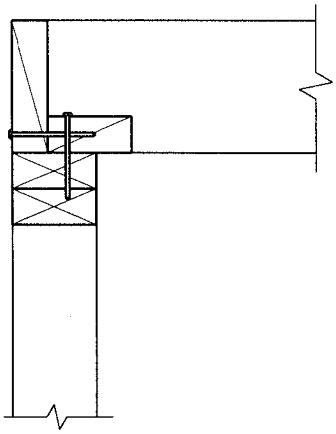


Figure C802-6. *Shear transfer utilizing face nailing at blocks*

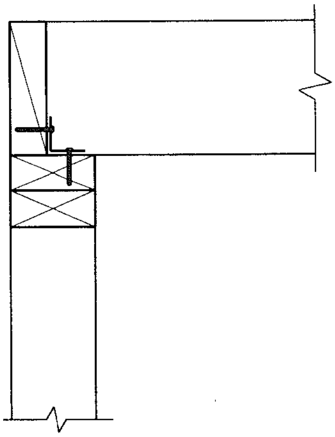


Figure C802-7. *Shear transfer utilizing framing anchors*

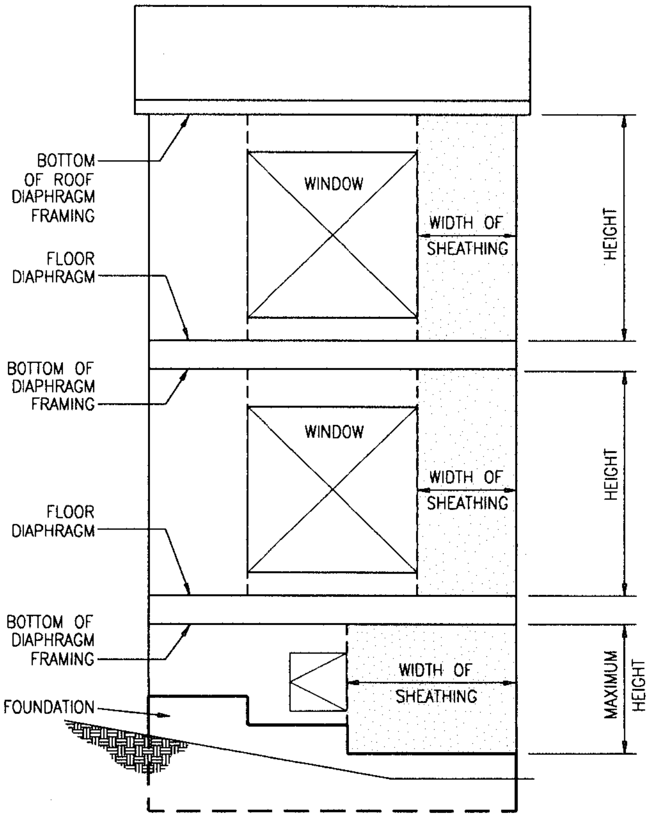


Figure C802-8. *Height of vertical diaphragm*

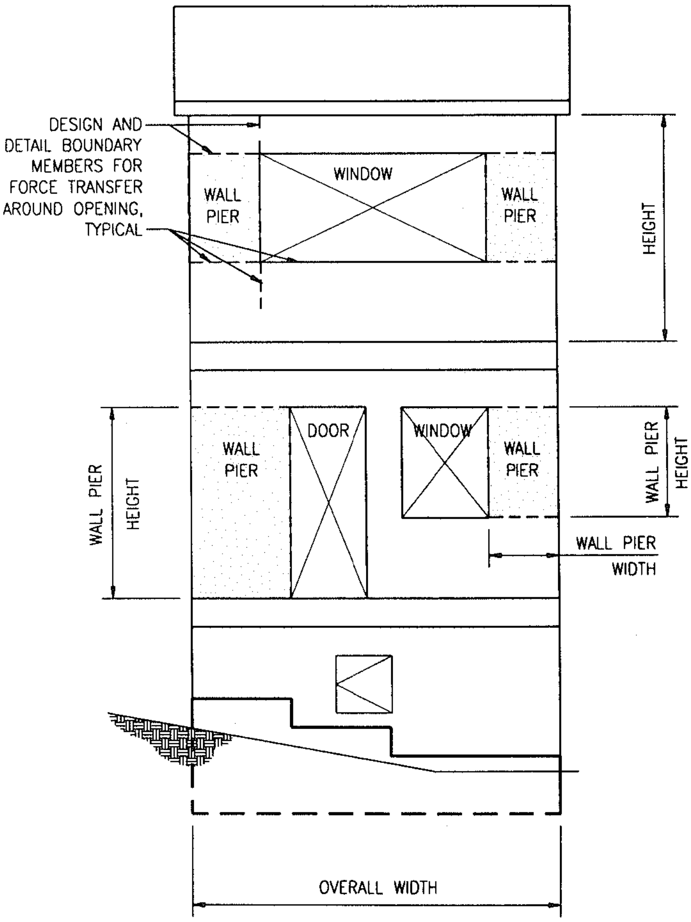


Figure C802-9. Height of vertical diaphragm with openings

CHAPTER 9

Other Materials

C901 General Requirements

C902 Recommended Modifications to 1994 UBC

None.

Additional Commentary

C903 Other Sheathing

Tests [Gray and Zacher, 1985; Rihal, 1985] have shown that reversed cyclic loading, representative of moderate to major earthquake response, can cause a significant degradation in the static one-way shear capacity of gypsum sheathing. Previous allowable stress values were based on one-way loading tests. The degradation of capacity effects are now represented by a reduction factor of one-half of the static energy.

References

Gray, R.G. and Zacher, E.G., 1985. "Dynamic Tests of Wood framed Shear Panels," *SEAOC Convention Proceedings*. Structural Engineers Association of California, Sacramento, California.

Rihal, S.S. and Granneman, G. 1985. *Dynamic Testing of Wood-Framed Building Partitions*. California Polytechnic State University, San Luis Obispo, California, Report ARCE-85-1. Prepared for National Science Foundation, Washington D.C., Grant no. CEE-81-17965.

APPENDIX A

Engineering Implications of the 1994 Northridge Earthquake

Northridge Commentary Committee

William T. Holmes (SEAONC), Chair
John M. Coil (SEAOSC)
S. K. Ghosh (PCA)
Ronald O. Hamburger (SEAONC)
Charles Kircher (SEAONC)
Ronald F. (Rawn) Nelson (SEAOSC)
C. Mark Saunders (SEAONC)
Edwin Zacher (SEAONC)
Theodore Zsutty (SEAONC)

APPENDIX A

Engineering Implications of the 1994 Northridge Earthquake

Introduction

The magnitude 6.7 M_w Northridge earthquake of January 17, 1994 provided a significant test for many buildings designed in accordance with these Recommendations. Although the strong ground shaking was of relatively short duration, spectral values in broad period ranges at many sites exceeded the Blue Book design spectrum. The performance of these tested buildings, both good and poor, must be carefully studied to determine engineering implications, formulate any revisions to these Recommendations that may be appropriate, and prepare improved designs in the interim.

Overall impressions of damage patterns or the performance of a single building may not be sufficient to justify changes in recommendations that will affect the design of all future buildings. The criteria used to determine the appropriateness of changes to seismic design recommendations should include the following considerations:

1. **Characteristics of the Ground Motion.** The exact motion at any one site is not known unless the site or building is instrumented. The possibility that extraordinary damage was caused by unusual ground motion or motion particularly tuned to a structure must be considered. The effects of duration are poorly quantified, but the acceptability of performance for any ground motion intensity should consider this factor.
2. **Expected or Acceptable Performance for the Intensity of Ground Motion Experienced.** Given that damage is expected for moderate and major levels of ground motion, the extent of actual damage should be measured against expectations before considering changes to design recommendations.
3. **Damage to Older Buildings.** Buildings designed to earlier versions of these Requirements should be carefully screened and considered separately. This performance may include effects of multiple deficiencies and therefore not be applicable as justification for specific changes to current recommendations.

4. **Quality of Construction.** Consider how well the current Requirements were implemented and whether improvements can be most efficiently be made by more stringent design requirements or better implementation of the current ones. The quality of design, construction, and inspection of certain types of wood construction, such as apartment buildings and strip malls, was found to be unacceptable in many instances.

Review and analysis of the results of any damaging earthquake is educational. Damage or damage patterns which, for one reason or the other do not warrant code changes, should be flagged so designers can consider individually whether to change their design practice with respect to structural systems, detailing, or interaction with the construction process. This appendix has been added to address concerns that may not yet have resulted in changes in formal design recommendations, but which should be considered in designs immediately. Additional information can be found in the Findings and Recommendations of the City of Los Angeles/SEAOSC Task Force on the Northridge Earthquake [LA/SEAOSC, 1994].

The primary design issues resulting from the Northridge earthquake include:

1. Concern regarding whether the damage and resulting economic losses to be expected using current Blue Book performance goals is generally acceptable.
2. Intensity and characteristics of the ground motion, particularly in comparison with standard design spectra, possible near field effects, and possible effects of vertical accelerations.
3. Performance of wood frame buildings, particularly gypsum board and stucco shear walls, and the performance of certain commonly used hardware.
4. Collapse of several concrete parking garages and the potential lack of deformation compatibility in many other concrete buildings.
5. Failure of wall diaphragm ties or cross building ties in stiff-walled buildings with flexible diaphragms—most notably tilt-ups.
6. The surprising failure of many beam-column joints in steel moment resisting frames, and the brittle fracture of certain steel elements in other situations.

Each of these areas of concern will be discussed separately.

Re-Evaluation of Blue Book Performance Goals

The well-known three-level performance goals found at the beginning of the Commentary and repeated below have served SEAOC well for nearly 40 years, particularly for the continuing development of code provisions to prevent collapse and protect life safety:

> "Structures designed in conformance with these Recommendations should, in general, be able to:
>
> 1. Resist a minor level of earthquake ground motion without damage.

2. Resist a moderate level of earthquake ground motion without structural damage, but possibly experience some nonstructural damage.
3. Resist a major level of earthquake ground motion having an intensity equal to the strongest either experienced or forecast for the building site, without collapse, but possibly with some structural as well as nonstructural damage."

> "It is expected that structural damage, even in a major earthquake, will be limited to a repairable level . . . ."
>
> ". . . In order to fulfill the life safety objective of these Recommendations, there are requirements that provide for structural stability in the event of extreme structural deformations . . . . While damage to the primary structural system may be either negligible or significant, repairable or virtually irreparable, it is reasonable to expect that a well-planned and constructed structure will not collapse in a major earthquake."

However, there are growing concerns within SEAOC about whether the current elastic equivalent lateral force seismic code format can be used to reliably predict performance over the full range of ground motions indicated, except in the simplest situations. Further, if performance at several levels of ground motion becomes more an integral part of the code, the above goals probably need complete redefinition.

Perhaps more important, damage in the Loma Prieta and Northridge earthquakes has caused the public to question whether these traditional code performance goals are acceptable. The public is more aware of the costs of both the repair of damage and lost use of buildings, and concern may be shifting beyond life safety to economic considerations. From a public policy standpoint, there is also interest in minimizing the demand for emergency shelter after earthquakes by attempting to provide performance that will yield fewer (or no) uninhabitable housing units.

One of the most difficult aspects of the multi-level performance description is the characterization of ground motion. Was the ground motion level at a given site minor, moderate, or major? Ground motions at some sites during the Northridge earthquake had spectral values among the largest ever recorded, but the duration of strong shaking was one-third to one-half of what is projected in other circumstances in Zone 4. Should damage be expected in accordance with performance level 2 (moderate ground motion) or 3 (major ground motion)? Prior to the current interest and open discussion about acceptable performance, the Blue Book performance goals have been used as a double-edged sword. On the one hand, any structure that did not collapse was considered to have met the goal (when compared with level 3), regardless of the level of damage or severity of ground motion. On the other hand, identified patterns of damage were almost always countered with code changes, regardless of the level of damage or severity of ground motion (apparently using level 2 as a goal).

Considering damage levels, it is becoming well recognized that, because of expected nonlinear structural behavior, deformations should replace forces as the key parameter. In order to improve damage predictability, target damage limitations must be linked to deformations and actual deformations must be better estimated.

In the future, as our ability to both define probable future ground motions and predict damage improves, more meaningful damage levels must be identified and coupled with more definitive ground motions so that the success or failure of code provisions can be more accurately gauged.

In current designs, if project performance objectives are set more definitively than those in these Requirements, the nonlinear behavior of the structure and the deformation demand of the expected or design ground motion should at least be considered.

Performance of Wood Frame Buildings

The vast majority of structures subjected to the Northridge earthquake were of wood frame construction. With the exception of a few notable cases, the structures generally withstood the earthquake without collapse and for the most part without extensive structural damage.

A study performed for the Department of Housing and Urban Development analyzed a random sample of several hundred wood framed structures within a 10-mile radius of the epicenter [NAHB, 1994]. Two groups of structures were analyzed: single-family detached properties (SFD), and single-family attached and multi-family low-rise properties (SFA/MFLR). The SFD results indicated remarkably low levels of damage, with 90 percent of the structures receiving no foundation damage, 98 percent receiving no wall framing damage, and 99 percent receiving no roof framing damage. The SFA/MFLR data was found to be inconclusive because of sampling problems. However, the data show a general trend of low damage occurrence, which is in general agreement with the Los Angeles Department of Building and Safety's inspection database. It was noted that much of the MFLR damage was due to a soft or weak story.

Multi-story structures with soft or weak first stories (generally resulting from wood framed parking levels) performed poorly, with many experiencing either a collapse of the first story or a gross residual horizontal deflection that resulted in a questionable ability to economically repair the building. Hamburger [1994] has estimated that approximately 200 structures either collapsed or had gross residual soft story deflection. The collapse of the three-story Northridge Meadows apartment building resulted in the loss of 16 lives [NIST, 1994]. Extensive cosmetic damage occurred throughout the area to wood frame structures in the form of cracking of interior wall finishes, and exterior plaster (stucco) cracking. In many buildings, these finish materials were also used as structural elements for resisting lateral forces. The cracking and damage experienced thus has not only resulted in cosmetic distress, but has compromised the lateral load resisting system of the structure.

When the stucco wall is erected, it is backed with a wire-mesh/building paper attached to the stud framing with power-driven fasteners. In many cases, the wire mesh was installed flush with the paper, providing little bond with the stucco. Two types of failures are noted by Hamburger [1994]: staples used to connect the stucco wall pulled out of the framing, and the stucco itself pulled away from the mesh.

Extensive cracking of finish materials on residential buildings has resulted in the need for costly repairs to many buildings not considered to be significantly damaged structurally. The extent of nonstructural damage and associated cost may well be much more extensive than the general public had anticipated.

Narrow plywood walls, primarily in the first story of multi-story buildings, experienced significant lateral displacement and incipient structural failures were observed [Andreason and Rose, 1994]. Hamburger [1994] reports on splitting of end studs at holddown bolt locations, in part because of inadequate stud size and mislocation of the holddown.

Many deficiencies in the construction of shear panels were noted. The use of sinker nails in lieu of the specified common nails, the occurrence of uncorrected shiners, and over-driven nails were causes of significant reductions in the strength of panels. These deficiencies occurred in spite of the inspection procedures of, arguably, one of the best municipal building departments in the nation. It should be noted that these deficiencies did not occur in wood frame school buildings where continuous inspection is required.

Many problems with sill bolting were observed due to poor construction practices. Anchor bolts were not properly located and were bent to fit into holddowns. This led to damage when these bolts straightened in tension. Many bolts were placed too close to the edge of the concrete and failed due to concrete spalling. Cripple wall failures were limited, primarily because most homes in the area are built slab-on-grade.

A design issue recently brought to light is the common assumption for light frame construction that the load carried by the various shear walls is directly proportional to their length. Narrow shear walls, those with height-to-width ratios greater than two, have been found to be much more flexible than assumed, even when properly constructed, due to the bending or overturning effects. Thus, these walls are not as effective in resisting earthquake loads as previously assumed. The longer walls, where present, will take a much higher portion of the load, and the horizontal distortion will be much larger than anticipated. These larger-than-anticipated deflections or drifts can result in more damage to nonstructural elements.

Recent testing of narrow shear walls [Shepherd and Allred, 1995] also indicate some concern with holddown connections at the ends of the walls. Observed failures included splitting of the wood post at the bolt holes, and substantial rotation and elongation of the holddowns. Sliding of sill plates after the anchor bolts split the wood was also observed. The American Plywood Association (APA) is also studying this issue. Early results of the APA tests were reported by [Commins and Gregg, 1994], and indicate that plywood nailing and holddowns distorted significantly, the National

Design Specification (NDS) bolt spacing values may be inadequate, and code deflections were exceeded when code forces were applied. Unfortunately, though the testing was cyclic, displacements were not taken to the levels that might be experienced during an actual earthquake, but rather to levels near the much lower values that might be used in code design [SSC 94-08].

As a result of a study of the performance of wood framed buildings by a task force of the City of Los Angeles and the Structural Engineers Association of Southern California, a series of recommendations for adoption by the City of Los Angeles, as it relates to wood frame buildings, has been made. Some of the major items in the report and the Emergency Enforcement Measures that the Los Angeles City Building Department has adopted are as follows:

1. Reduction of the allowable shear values for stucco and drywall to approximately half of that previously allowed by the code. Restrict the use of staples for attaching wire lath in stucco and restrict the height-to-length ratio for walls used for lateral resistance to no more than 1:1. In addition, restrict these walls to one-story buildings or the upper stories of multi-level buildings.
2. For plywood shear walls, reduce the code values by approximately 25 percent and restrict the height-to-length ratio to 2:1 in lieu of the previous 3.5:1 ratio. For heavily loaded walls, the width of members at the boundary and plywood joints would have to be at least 3x members.
3. Require that the size of holes for bolts at the hold-down anchors be verified to be no more than 1/16 inch oversize and that the hold-down bolts be tightened properly.
4. Require that column deflection in wood buildings be limited to 0.005 H (where H is the height of the column) and that design of cantilever columns be based on an equivalent buckling factor $K=2$.
5. Diaphragm rotation for the distribution of lateral forces will not be allowed.
6. Clearly show lateral force resisting systems on plans and elevations, and clearly show detail references of all shear walls on the plans.

Performance of Concrete Buildings

Nonductile reinforced concrete frame buildings allowed prior to the 1973 UBC did not perform well in the Northridge earthquake, as was only to be expected [NIST, 1994]. The seven-story Barrington building provided one of the worst examples of such nonductile building performance. The Kaiser Permanente office building in Grenada Hills suffered collapse of the second story throughout the length of the building and collapse of the end bays throughout the full height. The joints completely failed because of grossly inadequate confinement reinforcement. The structural system for the Bullock's department store at the Northridge Fashion Center, which collapsed, consisted of lightweight concrete waffle slabs supported by circular columns with a few poorly connected and discontinuous shear walls that were added as a retrofit shortly after the 1971 San Fernando earthquake [SEAOC, 1994; SSC 94-06].

Structures with shear walls as a primary lateral load resisting system performed well with respect to life safety and prevention of collapse. However, some shear walls showed sliding along horizontal construction joints, diagonal flexure-shear cracking, and significant damage to coupling beams and short piers between openings.

Post-1973 reinforced concrete buildings generally performed well, with some noted exceptions [SSC 94-06] including some parking structures [SEAOC, 1994].

There were many concrete parking structures close to the epicenter and farther away, both old and new, that came through the earthquake with very little or no damage. As a class, however, parking structures (cast-in-place, as well as precast) fared considerably worse than other concrete buildings, even though most were post-1973 structures.

Design Implications for Concrete Buildings

The following observations concerning the seismic design implications of the Northridge earthquake for concrete buildings should be noted.

Vertical Acceleration. Immediately following the earthquake, some of the damage was blamed on vertical acceleration, which was supposedly exceedingly high at many locations. Recorded peak horizontal accelerations were plotted by Moehle [EERC, 1994] against the peak vertical accelerations recorded at 38 stations. At a few locations vertical accelerations were nearly equal to the horizontal accelerations, and at one station the peak vertical acceleration exceeded the peak horizontal acceleration. However, in general, peak vertical accelerations were about two-thirds the peak horizontal accelerations, as would normally be expected. Priestley [UCSD, 1994] concluded that vertical accelerations played no significant role in the damage sustained by seven highway bridges that failed in the Northridge earthquake. However, as the absolute value of vertical acceleration increases, its potential for significant interaction with both coincident lateral motion and gravity load increases, particularly in structures that may have amplifying vertical response.

Section 105.10 of these Requirements requires explicit consideration of vertical acceleration in the design of horizontal cantilever components as well as horizontal prestressed components. Whether explicit consideration of vertical acceleration needs to be expanded beyond that remains unclear at this time.

Structural Systems and Deformation Compatibility. These Requirements formally recognize only four different structural systems for reinforced concrete buildings:

- Moment-resisting frame system
- Bearing wall system
- Dual (shear wall-frame interactive) system
- Building frame system

All other structural systems for reinforced concrete buildings fall under the category of undefined systems.

In recent years, there has been a proliferation of reinforced concrete structures, both parking garages and commercial buildings, that combine ductile and nonductile frames in the same building. Typically, two ductile frames on two parallel faces of a building are relied upon for the entire lateral resistance in the direction parallel to those faces. All other frames spanning in the same direction are designed for gravity only. Are these buildings to be designed as moment-resisting frame systems, or as undefined systems? The conservative answer is probably the latter. In view of the popularity of this system, guidance needs to be developed as to its design.

These Recommendations specifically require that any structural element designated as not part of the lateral load resisting system of a building in a moderate or a high seismic zone be designed to be able to sustain factored gravity loads imposed on it under a lateral displacement equal to $(3R_w/8)$ times the lateral displacement of the lateral load resisting system elastically computed under the code-prescribed seismic forces. This deformation compatibility requirement is crucial to the safety of the building frame system, where lateral loads are resisted by shear walls and frames carry gravity loads only. It is even more crucial to the safety of buildings that combine a small number of ductile lateral load resisting frames with a significant number of nonductile frames designed for gravity only. Unfortunately, the deformation compatibility check is seldom properly carried out by design engineers, and even more seldom enforced by local jurisdictions. Deformation compatibility requirements have been made more stringent and more enforceable in the 1996 Accumulative Supplement to the UBC (see Appendix C1-AppE).

The UBC further requires that a column that is not part of the lateral force resisting system in Seismic Zone 3 or 4 be detailed like a column that is part of the resisting system in Seismic Zone 2 (1994 UBC §1921.7.2.2). In the absence of proper deformation compatibility checks, this required level of detailing is not sufficient and needs to be made more stringent. This has been done in ACI 318-95 as well as in the 1996 Accumulative Supplement to the UBC (see Blue Book Section 402.13).

Section 1909.3.4.2 of the 1994 UBC appears to allow the use of overstrength, nonductile columns even as part of the lateral force resisting system. That section is confusing, may lead to unsafe designs, and has been deleted from ACI 318-95 as well as from the 1996 Accumulative Supplement to the UBC. This particular section has been corrected by the Blue Book Section 402.7.

Precast Prestressed Concrete. Specific code requirements are needed for precast, prestressed concrete structures in high seismic zones. Such specific provisions will replace the current vague requirement that precast, prestressed concrete structures be equivalent to monolithic concrete structures in terms of strength and toughness. The interpretation, implementation, and enforcement of the current requirement has been nonuniform for obvious reasons. The Building Seismic Safety Council has approved a set of specific provisions for precast, prestressed concrete structures in regions of high seismicity, which appear in the 1994 edition of the NEHRP Recommended Provisions [BSSC, 1994]. The NEHRP requirements, applicable to precast concrete frame structures emulating monolithic construction, with significant additions and modifications, have been recommended by the ICBO Lateral Design Code Development Committee for inclusion in the 1997 UBC.

Continuous Load Path. Section 108.2.4 of these Requirements contains "Ties and continuity" requirements. Section 108.2.5 contains "Collector Elements" requirements that read: "Collector elements shall be provided that are capable of transferring the seismic forces originating in other portions of the building to the element providing the resistance to those forces" (see Commentary Section C108.2.5). Section 108.2.8 contains requirements for "Anchorage of Concrete or Masonry Walls." In spite of all of this, shear walls in more than one precast parking structure that suffered partial collapse were observed to be relatively intact. The most informed speculation is that the collector elements contained insufficient reinforcement, which yielded early on, rendering the elements unable to transmit the lateral forces to the shear walls. There may have been other modes of diaphragm failure. This is a breakdown of the continuous load path and needs to be investigated.

Cast-in-Place Topping Acting as Diaphragm. An area of weakness in modern precast parking structures is the flexibility of the thin cast-in-place topping slab that forms the horizontal floor and roof diaphragms and the chords and collectors for these diaphragms. The UBC currently requires these topping slabs to be designed to act as diaphragms independently of the precast elements. The use of a composite design of the topping together with the precast elements to act a as diaphragm may very well be preferable. Details of chord reinforcing in thin topping slabs need improvement [SSC 94-06].

Damage Control. The Northridge earthquake has once again pointed out the desirability of including reliable damage control criteria in seismic provisions. Whether the current limitation on interstory drift computed elastically under code-prescribed seismic forces provides such damage control is highly questionable. This is particularly so because the stiffness assumptions under which the drift computations are to be made are not specified (see Commentary Chapter 1, Appendix 1E). The code-prescribed design seismic forces are dependent on the structural period, which in turn also depends on stiffness assumptions. However, these stiffness assumptions may not be explicit if the approximate formulas of these Recommendations are used for period computations or if the limitation is invoked, which restricts the design force level to be no less than 80 percent of the design force level based on the approximate period of the code (see Commentary Sections C105.8.2, C105.8.3, and C106.3).

Performance of Tilt-Ups

The Northridge event was the first since the 1971 San Fernando earthquake to subject a large number of tilt-up and masonry buildings to ground motions as strong as those projected for the most severe events. Within the San Fernando Valley, which encompasses the areas most strongly shaken by the Northridge earthquake, there are an estimated 1,200 tilt-up buildings [Brooks, 1994], most with panelized plywood roofs. An estimated 400 of these buildings [Deppe, 1994] within the administrative limits of the City of Los Angeles experienced severe damage, including partial roof collapses and, in some cases, collapse of perimeter walls. Additional similar damage, not included in the above statistics, occurred outside the City of Los Angeles. Many of the damaged buildings were designed to recent editions of the UBC. While no lives were lost as a result of this performance, the damage sustained by many buildings was clearly life-threatening.

Primary structural failure occurred at the out-of-plane connections between the perimeter masonry or concrete walls and the panelized plywood roofs. Many failures occurred in buildings provided with specially detailed wall ties, mandated by the UBC in every edition since 1973. Nearly every conceivable type of wall tie connection experienced failure, including catalog hardware supplied by timber connector manufacturers, as well as custom designed and fabricated details. Wall tie failures included: shear cone pullout of anchors within the masonry or concrete walls; splitting of timber members at bolted connections; and fracture of light gauge metal hardware, particularly at locations of bends, swivel joints, and bolt holes.

These buildings are classical "box" type structures with flexible diaphragms. Research on instrumented buildings following the 1984 Morgan Hill [Celebi, 1989] and 1989 Loma Prieta Earthquakes [Bouwkamp, 1991] indicates that the dynamic response of structures with predominantly solid walls is dominated by the diaphragms, with initial periods in the range of 1 to 3 seconds. Walls aligned parallel to the direction of ground motion behave in a relatively rigid manner and do not significantly amplify ground motion (recordings indicate less than 40 percent amplification of peak ground acceleration). However, the diaphragms do significantly amplify the motion transmitted by the walls. For low levels of ground motion, peak accelerations recorded at diaphragm centers are often three times those recorded at the ground. For strong ground motion, the diaphragms respond with less amplification. Diaphragm shear strength appears to be the limiting factor. Once a diaphragm yields in shear, it can not deliver any more force to the mass supported by the roof, thereby limiting the amount of acceleration that can be induced.

Typical diaphragms in UBC Zone 4 are designed with code level shear strengths equal to approximately 20 percent of the supported weight. Published test data on plywood diaphragms [Elliot, 1980] indicates that code level design shears have a factor of safety under monotonic loading conditions ranging from 3 to 4. Therefore, roof diaphragms without significant overstrength can deliver an effective shear equal to approximately 80 percent of the diaphragm's supported weight. Assuming diaphragm accelerations vary from approximately Z at the edge to $3Z$ at the center, the average roof acceleration is approximately $2Z$. Equating the ultimate diaphragm shear ($0.8W$) with the average roof acceleration times the effective mass ($2ZW$), such a roof would reach its ultimate strength when subjected to a peak ground acceleration of 0.4*g*. The maximum acceleration that would ever be expected at the center of the such roof diaphragms would be three times this amount, or a maximum of 1.2*g*, whether or not the actual ground motion exceeds a peak acceleration of 0.4g. Stronger diaphragms, however, could produce larger accelerations.

Given this data, ultimate demands on out-of-plane roof-to-wall connections can be estimated. For a one-story unit strip of wall, with height *h* and weight *w* lbs./ft., subjected to acceleration "*Z*" at its base and "3*Z*" at its roof support point, simple calculations indicate a maximum wall anchorage (force) at the roof level of 1.167 *Zwh*. This force is directly proportional to the strength of the ground motion, represented by the coefficient *Z*, but is also limited, as previously described by the shear strength of the diaphragm. For roofs having a shear strength conforming to, but not significantly exceeding, code requirements, the maximum force that can be delivered at the top of the wall can be calculated by substituting the code value for *Z* in the above expression. For Zone 4, this yields a maximum wall anchorage force of 0.467 *wh* pounds/foot of wall. Stronger diaphragms would be able to produce larger forces.

Based on failures in the 1971 San Fernando earthquake, codes since the 1973 UBC have required positive direction connections of walls to diaphragms, and continuous diaphragm ties to distribute anchorage forces into the diaphragms. In the 1994 UBC, anchorage forces are governed by the equation:

$$F_p = Z\, I\, C_p\, W_p$$

To account for diaphragm amplification effects, anchorage forces in the middle half of the diaphragm are increased an additional 50 percent. For Zone 4, this results in anchorage design forces of 0.45 *wh* in the middle half of the diaphragm, and 0.30 *wh* elsewhere. In general, hardware used for wall anchorage incorporates a minimum factor of safety of 1.7 against failure. This results in ultimate connection strengths of 0.77 *wh* and 0.51 *wh*, respectively, in the two diaphragm regions.

It was previously shown that actual peak demands on wall anchors at the centers of roof diaphragms that meet, but do not exceed, code requirements is 0.467 *wh*. For diaphragms with substantial overstrength, the excessive demands on wall anchors can be much larger. Long rectangular diaphragms commonly have substantial overstrength aligned with the long diaphragm dimension. In such cases, current design practice may result in anchors that are under-designed by as much as 100 percent. Inadequate wall anchor strength would not be a significant concern if anchors exhibited ductile behavior. Unfortunately, observed performance of common anchorage systems in the Northridge earthquake indicates that they do not.

As much as 40 percent of the total number of tilt-up and masonry buildings located in that portion of the City of Los Angeles that lies within the San Fernando Valley experienced severe, life threatening damage. The percentage of buildings experiencing such damage within the areas of strongest ground motion (some of it outside the limits of the City of Los Angeles) was much higher. Analytical evaluations indicate that the 1994 UBC requirements may inadequately protect buildings when actual ground motions approach or exceed the nominal peak accelerations represented by the zone coefficient Z. The problem appears to be most severe for buildings located on soft soil (S3) sites or within the near-field zone of the causative fault, due to the long period nature of motion experienced on such sites.

Changes were adapted into the 1996 UBC Supplement for the out-of-plane wall anchorages to flexible diaphragms for Seismic Zones 3 and 4. The changes (see Requirements Section 108.2.8.1) are:

1. All elements of the wall anchorage system are to be designed using C_p equal to 1.2 over the entire length of the diaphragm. This results in a minimum design force of $0.48wh$ in Seismic Zone 4. The load factor for the concrete embedment design is 1.7 (rather than 1.4). The design force for the working stress design of steel elements is increased by a factor of 1.7, or $1.7 \bullet ZC_p wh = 1.7 \bullet 0.4 \bullet 1.2wh = 0.82wh$ for Seismic Zone 4.
2. Eccentricity must be explicitly considered in the design of all elements.
3. Wood elements must have a minimum net thickness of 2 1/2 inches (3x rafters) if used as part of the anchorage system.
4. Wall pilaster can significantly affect the force distribution. Their configuration and details must provide for stable support at the maximum earthquake deformation level.
5. Anchorage using embedded straps must be attached to or hooked around reinforcement or otherwise terminated to effectively transfer the force from the strap to the reinforcing steel.

Performance of Steel Moment Frame Buildings

The Northridge earthquake caused damage to over 200 steel moment resisting frame buildings. The majority of this damage consists of fractures of the bottom flange weld between the column and girder flanges, although there have also been a large number of instances where top flange fractures occurred. In some cases, damage at these connections included tearing and cracking of the column flanges and/or web, and there were instances of cracking and complete fracture of heavy column sections. Tearing of the girder shear tab connection also occurred in a number of instances when flange welds were fractured. Buildings of 1 to 27 stories have been affected, with the majority in the range below six stories. Most of the damage occurred to structures of recent construction, although buildings up to 20 years old experienced weld fractures. The majority of the damaged steel frame buildings are located in the recently developed San Fernando Valley. Similar, but less extensive, damage has been found in buildings in Santa Monica and West Los Angeles.

Since data about the damage suffered by these buildings is still being collected, it is difficult to accurately determine the similarities in design and construction between steel moment frame buildings affected by the Northridge earthquake and those in other areas. It is likely that they are very similar, if not identical, to design and construction practices in other parts of the state and country.

Almost without exception, the connections that failed were of the type prescribed by paragraph 4F.1.b of the 1990 edition of the Blue Book. That type of connection was first specifically prescribed in the 1988 Blue Book and in the 1988 UBC, but similar connections have been in common use for about 20 years. The history of this type of connection (welded flanges/bolted web plate) dates originally to the work of Popov and Stephen in the early 1970s [Popov and Stephens, 1970]. This investigation, using W18x50 and W24x76 beams, compared the performance of all-welded connections to that of welded flange/bolted web connections. Although the investigation showed that the welded flange/bolted web connections did not perform as well as the all-welded, and that their performance was somewhat erratic, nonetheless these connections did develop substantial inelastic beam rotations, and were accepted by structural engineers as being adequate for seismic applications. The conclusions drawn from this investigation, coupled with the basic economy of the connection and an increasing development of trust in welding, led to such widespread acceptance by structural engineers and fabricators that the welded flange/bolted web connection became fully accepted and was written into the code. Subsequent tests by Tsai and Popov (1988), and by Englehardt (1993), further demonstrated the variability of performance of these welded flange/bolted web connections.

Despite the lack of complete reliability of the connection as demonstrated in the tests, the number of failures at Northridge would have to be considered as quite unexpected. The following factors may have contributed to this poor performance:

1. Unreliability of material properties, particularly excessive strength of A36 beams and girders.
2. Poor execution and quality control of welding.
3. A basic joint configuration not conducive to ductile behavior.
4. Web connections with bending strength less than that of the web itself.
5. Unreliable through-thickness strength of column flange material.
6. Large member sizes and large welds.
7. High residual stresses caused by fit-up sequence of highly restrained joints.
8. Possible effects of unusual earthquake characteristics such as large vertical accelerations.

A number of groups concerned with the design and construction of steel moment resisting frame buildings have begun work on determining the causes of the damage. With initial funding from the California OES, the SAC Joint Venture, consisting of SEAOC, the Applied Technology Council (ATC), and the California Universities for Research in Earthquake Engineering (CUREe), was formed to begin research to resolve the problem. The SAC Joint Venture developed a two-phase program. The main intent of Phase I was to provide interim guidelines for the immediate post-Northridge problems of identifying damage in affected buildings and repairing the damage using methods that had been proven capable by testing. Phase I also included documentation of the best guidance available for design of new steel moment frames. Phase II consists of a long-term program of research to understand conditions that lead to the premature connection fractures and to develop sound guidelines for seismic detailing of improved or alternative connections for new buildings, as well as reliable repair and retrofitting concepts for existing WSMF structures. Phase I was funded by OES and FEMA and has been completed. Phase II is being funded by FEMA and is expected to be completed in 1998.

The major conclusions of the SAC Phase I effort is contained in the document entitled *Interim Guidelines: Evaluation, Repair, Modification and Design of Welded Steel Moment Frame Structures* (FEMA, 1995). A companion set of documents, including the research reports and state-of-the-art papers, is also available. A large amount of detailed information is included in the *Interim Guidelines*, but the main conclusions were:

- No single cause for the failures was noted, but rather the problem was likely a combination of many factors.
- Even though there were no collapses of WSMF structures in this earthquake, such collapses should not be ruled out. However, well-constructed WSMF should be considered to pose less risk of collapse than many other existing buildings, including unreinforced masonry, nonductile concrete frames and other concrete buildings with inadequate deformation capacity, and certain tilt-up and precast structures.
- Because of the potential for hidden damage, a sample of connections in all WSMFs subjected to strong ground motion should be inspected. Inspection guidelines are included.
- For limited levels of damage, welds may be repaired to their original configuration using controlled procedures specified in the guideline. It is recommended that for more serious damage, modification of the connectors or of the entire lateral force system be considered. Conceptual modification details that have shown promise in limited testing are included.
- In general, design of connections for new buildings should be based on project-specific tests. Exceptions to this recommendation can be made under certain circumstances, such as the availability of similar tests, and qualified third-party review of the design. A summary of connections types tested to date is given.

Guidance is also included on specifications for welding, quality control, and nondestructive testing.

Another issue for structural engineers to consider is the shear connections of nonmoment frame girders to columns. Considering that many web shear tabs cracked under the rotations imposed on them after the flange welds failed, and recognizing that the nonframe connections must undergo similar rotations, the advisability of beams or girders being rigidly connected to shear tabs by slip-critical bolts or by welds must be evaluated.

In the fall of 1994, an emergency code change was made to the 1991 UBC that deleted provisions prescribing requirements for beam-column joint connections, the UBC equivalent of the 1990 Blue Book Section 4.F.1.b. The section was replaced by performance oriented requirements that require considerable interpretation. Current interpretation is contained in SEAOC Interim Recommendation Number 2 published in January, 1995.

Strong Ground Shaking Recorded During the 1994 Northridge Earthquake

Strong ground shaking was recorded during the magnitude $M_w = 6.7$ 1994 Northridge earthquake at sites located close to fault rupture. At these near-source sites, ground motion response spectra were often significantly larger than the Seismic Zone 4 design spectrum (i.e., the normalized response spectrum shown in Figure 104-2 for the soil profile type of interest, anchored to an effective peak ground acceleration of 0.4g). While somewhat stronger than that expected for a $M_w = 6.7$ event, recorded near-source ground shaking was consistent with both theoretical and empirically based predictions of ground motion for large magnitude earthquakes.

The design spectra of these Recommendations are representative of ground shaking at sites located about 10 to 15 km from the fault rupture of a large magnitude earthquake. For base isolated structures located within 15 km of an active source, these Recommendations prescribe a near-field coefficient, N (see Table 152-3), that is used to increase seismic demand based on earthquake magnitude and the closest distance to fault rupture. The Ad Hoc Ground Motion Subcommittee of the Seismology Committee has developed a similar near-field factor for design of both isolated and conventional (fixed base) structures. This factor will appear in future editions of the SEAOC Blue Book and is scheduled for probable incorporation in the 1997 UBC.

Background

The Northridge earthquake of January 17, 1994 was centered in the San Fernando Valley. The epicenter was about 1 mile south-southwest of Northridge, or about 20 miles west-northwest of downtown Los Angeles. The moment magnitude of the earthquake is $M_w = 6.7$.

Analyses of aftershock data indicate that the mainshock fault rupture plane covered an approximately rectangular zone extending about 15 km west-northwest from the epicenter and about 15 km to the northwest-northeast. The fault rupture plane slopes upward at about 45 degrees from a depth of about 18 km at the mainshock's focus to within about 5 km of the surface. The fault rupture plane slopes upward to the northeast so that areas closest to the fault rupture lie roughly along Interstate 5 between Sylmar and Newhall.

Strong ground motion was recorded at a number of stations located within 10 to 15 km of the fault rupture. These recording stations include:

California Strong Motion Instrumentation Program (CSMIP)

- Arleta (Nordhoff Avenue Fire Station)
- Sylmar (Los Angeles County Hospital)
- Newhall (Los Angeles County Fire Station)

National Strong-Motion Program (NSMP)

- Sepulveda V.A. Hospital (United States Department of Veterans Affairs)
- Jensen Filter Plant (Metropolitan Water District of Southern California)

Los Angeles Department of Water and Power Strong-Motion Network

- Rinaldi Receiving Station
- Sylmar Converter Station (SCS)
- Sylmar Converter Station East (SCSE)

A summary of ground motion recorded at the stations listed above is provided in Table AppA-1. Data presented in Table AppA-1 are taken from CSMIP [CDMG, 1994], NSMP [USGS, 1994] and LADWP [Lindvall et al., 1994] reports. Estimates of the closest distance to fault rupture are based on the reported location of the instrument and the approximate location of the mainshock fault plane. Estimates have been rounded to the nearest 5 km, reflecting imprecise knowledge of these distances. The data include only records from free-field stations (or instruments located at the ground floor of buildings when free-field records are not available). Records from the upper floors of buildings or from dams are not included.

The records summarized in Table AppA-1 are a fairly complete and representative sample of the strongest shaking recorded during the 1994 Northridge earthquake. One notable omission is the exceptionally strong shaking recorded at Tarzana-Cedar Hills Nursery [CDMG, 1994]. This station recorded ground motion with peak ground accelerations greater than 1g in each of the three orthogonal directions. Very large accelerations were recorded for at least 10 seconds. Aftershock evaluation verified instrument functionality; however, comparisons with other nearby instruments revealed highly localized amplification at this site. The Tarzana record should not be ignored, but the record should not be considered representative of ground motion recorded during the event.

Near-Field Ground Motion—Horizontal Direction

Since 1992, the Ad Hoc Ground Motion Subcommittee of the SEAOC Seismology Committee has been considering refinements to ground motion criteria. Scope and status of Subcommittee activities have been reported annually at the SEAOC convention [Kircher et al., 1993]. The primary objective of the Subcommittee is to develop seismic criteria that better reflect actual ground shaking expected at building sites. The Subcommittee has been developing a near-field factor for sites located near earthquake sources. This factor would be incorporated into design values required for equivalent lateral force (static) analysis design and the seismic criteria required for dynamic analysis. At present, a near-field term is required only for design of seismic isolated buildings.

Ground motion recorded during the 1994 Northridge earthquake at sites near the fault rupture provide additional and compelling evidence for design consideration of ground motion beyond the level specified by current SEAOC seismic design criteria. As summarized in Table AppA-1, horizontal components of ground acceleration recorded approximately 5 km from the fault rupture have instrumental peak ground acceleration (PGA) values of about 0.7g, on the average, in contrast to the 0.4g "effective" PGA of the spectrum recommended by SEAOC.

A more meaningful evaluation of near-source ground motion may be made by directly comparing response spectra. Such comparisons avoid issues related to inherent differences in instrumental and effective values of PGA, and provide better information on ground shaking at periods that most affect building response. Figure AppA-1 compares spectra recorded at three CDMG stations: Arleta, Sylmar, and Newhall. For each station, the spectra of the two horizontal components have been averaged together. This averaging tends to smooth the peaks and valleys of individual components (permitting easier comparison with the SEAOC spectrum), but eliminates biases between components due to directivity and other source-site effects. In most near-field records, fault-normal component of ground shaking tends to be significantly stronger than the fault-parallel component.

As shown in Table AppA-1, the Arleta station was the closest of the three stations to the epicenter (10 km), but the farthest from fault rupture (15 km). Both Sylmar and Newhall stations were farther from the epicenter, but significantly closer to the fault rupture (5 km). Comparison of the Arleta, Sylmar, and Newhall spectra reflects these differences in distance to fault rupture. Arleta is about one-half to three-quarters of the SEAOC spectrum at periods of interest, while the Sylmar and Newhall spectra are both about 1.5 to 2 times the SEAOC spectrum. Figure AppA-2 compares composite Sylmar/Newhall spectra with the SEAOC spectrum. Both the average and the envelope of the four spectra of individual horizontal components are compared. The average spectrum indicates that ground shaking at Sylmar and Newhall near-field sites typically exceeded the SEAOC spectrum by a factor of about 1.5 to 2 at virtually all periods of interest to building design (i.e., all periods between about 0.5 seconds and 3.0 seconds). The envelope spectrum suggests that motion in the strongest direction of shaking recorded at the critical station exceeded the SEAOC spectrum by about a factor of 2 to 2.5 at all periods of interest to building design.

Building proximity to fault rupture is an important parameter, since ground motion attenuates with distance. Figure AppA-3 illustrates ground motion attenuation from a fault in terms of a 1-second response. The trend in ground motion attenuation is similar for other periods. In Figure AppA-3, empirically based predictions of attenuation are based on the work of Boore et al. [1993], although the work of other researchers could have been used. Boore et al. uses a distance definition of the horizontal distance to the surface projection of the fault rupture, which is different than “closest distance to source,” but the points discussed below are still valid. Soil type is assumed to be Boore et al. soil type B/C, corresponding to medium soil site conditions (i.e., SEAOC Soil Type S2). Attenuation values are provided for two earthquake magnitudes, $M_w = 6.7$ and $M_w = 7.5$. In both cases, attenuation is shown by values that range from the “median” to “ + 1 sigma” levels of predicted response. The range of predicted values is large, reflecting the large scatter in measured ground motion.

As shown in Figure AppA-3, the empirically based predictions of the shaking level drop off significantly with distance. Superimposed on the empirically based predictions shown in the figure are response values obtained from 1994 Northridge earthquake spectra of six of the near-field records listed in Table AppA-1. Five of the records (Rinaldi, Newhall, Sylmar, SCS, and SCSE) are at about 5 km from fault rupture and the sixth record (Arleta) is at about 15 km. Also shown for reference is the value of the SEAOC spectrum for Soil Type S2 at a period of 1 second. This value is 0.6 g for Zone 4 and 0.45 g for Zone 3. When multiplied by the building's weight, these accelerations are the same as the design values required for equivalent lateral force design of a 1-second building, before reduction by the R_w factor. Within a given seismic zone, the SEAOC spectrum (and corresponding design values) do not vary with distance from earthquake source(s). As illustrated in Figure AppA-3, the SEAOC spectrum would be expected to underpredict ground shaking for sites located within about 10 to 15 km of fault ruptures. The near-field records of the Northridge earthquake confirm this expectation.

Near-Field Ground Motion—Vertical Direction

Vertical ground motion is assumed to be about two-thirds of horizontal ground motion, except at distances closer than about 10 km, where the peak vertical acceleration is approximately equal to the peak horizontal acceleration. In this sense, the vertical motions recorded during the Northridge earthquake are not exceptional. For free-field stations listed in Table AppA-1, the average vertical PGA is about 0.6*g*, in contrast to an average horizontal PGA of about 0.7 g. The important aspect of these values is that while both are large, the instrumental vertical PGA is over twice that corresponding to two-thirds of 0.4 g, the effective horizontal PGA of the SEAOC spectrum.

Large vertical PGAs, in and of themselves, would not necessarily be significant. Typically, the strongest portion of vertical motions tend to shake the site before the strongest portion of horizontal motions can excite the building. Close to fault rupture, however, the strong vertical and horizontal motions can act concurrently on buildings. Near-field records from the 1994 Northridge show significant correlation of vertical and horizontal ground shaking.

The most important aspect of evaluating the effects of vertical earthquake ground shaking on buildings is the determination of the vulnerability of building elements to such motions. Certain buildings have elements, such as the long, vertically flexible spans of a parking garage, that would be expected to be dynamically excited by vertical earthquake shaking. Spectra of the vertical component from a number of near-field locations show peak response in the range of about 1g to 2 g. These high response accelerations were found at periods from 0.1 to 0.5 seconds, the range of periods typical of vertical modes of vibration. This level of vertical earthquake loads, when combined with building weight, would generate short duration loads on elements as much as the supported weight acting upward or as much as three times the supported weight acting downward. Buildings that have significant design margin on vertical (beyond the weight of the building) can sustain high, but short-duration, vertical earthquake accelerations without failure. Elements of buildings located near fault rupture that do not have such design margin would be susceptible to vertically-induced earthquake failures.

Duration

The duration of strong ground shaking of the 1994 Northridge earthquake was relatively short, consistent with that expected for a magnitude $M_w = 6.7$ event. Teleseismic and strong motion data indicate that two main subevents occurred, separated in time by about 2 to 3 seconds (with the possibility of a smaller event 4 to 5 seconds later). These data suggest the duration of fault rupture was about 6 to 8 seconds, consistent with the 8- to 10-second duration of very strong shaking recorded at near-source sites. With respect to the strongest shaking recorded at sites located at about 5 km from fault rupture, the time histories typically include only one or two cycles of strong velocity pulses that most affect the long-period portion of response spectra.

The relatively short duration strong ground shaking likely reduced the effects of the very high demands placed on buildings located near the fault rupture. Buildings were required to sustain only a few cycles of extreme response and properly designed lateral force resisting systems of most building types generally performed well. Caution must be exercised in extrapolating these favorable observations to earthquakes of larger magnitudes, since the duration of strong shaking would be expected to be much longer.

References

Andreason, Kenneth and John Rose, 1994. *Northridge, California Earthquake: Structural Performance of Buildings in San Fernando Valley, California*, American Plywood Association: Tacoma, Washington, APA Report T94-5, March.

Boore, D.M., W.B. Joyner, and T.E. Fumal, 1993. "Estimation of Response Spectra and Peak Accelerations from Western North American Earthquakes: an Interim Report," U.S. Geological Survey Open-File Report 93-509.

Bouwkamp J., R.O. Hamburger, and J. Gillengerten, 1991. *Degradation of Plywood Roof Diaphragms Under Multiple Earthquake Loading*, CSMIP90.

Brooks H., 1994. *Tilt-Up and Earthquakes: A Post-Northridge Assessment.*

BSSC, 1994. *NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings*, Building Seismic Safety Council, Washington, D.C.

CDMG, 1994. California Department of Conservation, *CSMIP Strong-Motion Records from the Northridge, California Earthquake of January 17, 1994*, California Division of Mines and Geology Office of Strong Motion Studies Report No. OSMS 94-07, California Strong Motion Instrumentation Program.

Celebi, M., G. Bongiovanni, E. Safak, G. Brady, 1989. "Seismic Response of a Large Span Roof Diaphragm," *Earthquake Spectra.* Earthquake Engineering Research Institute, Oakland, California.

Commins, A. and R. Gregg, 1994. *Cyclic Performance of Tall-Narrow Shearwall Assemblies*, paper and presentation at 1994 EERI Annual Meeting, Pasadena, California, April 7-9.

Deppe, K., *Earthquake Performance of Tiltup, Masonry and Wood Frame Buildings,* 1994. Presented at the Joint Meeting of the Earthquake Engineering Research Institute and Seismological Society of America. April.

EERC, 1994. *Preliminary Report on the Seismological and Engineering Aspects of the January 17, 1994 Northridge Earthquake*, Report No. UCB/EERC-94/01, Earthquake Engineering Research Center, University of California, Berkeley.

Elliot, J.R., 1980. *Wood Diaphragm Testing - Past, Present, Planned*, in Proceedings of a Workshop on Design of Horizontal Wood Diaphragms. Applied Technology Council, Redwood City, California.

Engelhardt, M.D. and A.S Hussain, 1993. Cyclic Loading Performance of Welded Flange-Bolt Web Connections, *Journal of Structural Engineering*, ASCE, December 1993.

FEMA 267, 1995. *Interim Guidelines: Evaluation, Repair, Modification, and Design of Steel Moment Frames.* Report No. SAC 95-02.

Hamburger, R.O., 1994, *Lessons Learned in the Northridge Earthquake on Wood Frame Buildings*, Seminar Papers: Northridge Earthquake: Lessons Learned, SEAONC 1994 Spring Seminar, SEAONC: San Francisco, California.

Kircher, C. A., N.C. Donovan, L. Mualchin, J. Ragsdale, K.L. Benuska, M. Lew, V. Berger, and G. Magee, 1993. *An Acceptable Method for Characterizing Seismic Hazard, Proceedings 62nd Annual Convention.* Structural Engineers Association of California, Sacramento, California.

Lindvall, Richter, and Benuska Associates, 1994. *Processed LADWP Power System Strong-Motion Records from the Northridge, California Earthquake of January 17, 1994*, Report LRB 007-027. Prepared for Los Angeles Department of Water and Power by Lindvall, Richter, and Benuska Associates Los Angeles, California.

LA/SEAOSC, 1994. City of Los Angeles/Structural Engineers Association of Southern California, *Findings and Recommendations of the City of Los Angeles/SEAOSC Task Force on the Northridge Earthquake.* SEAOSC Seminar Notes, SEAOSC, 5360 South Workman Mill Road, Whittier, California 90601.

NAHB (NAHB Research Center), 1994. *Assessment of Damage to Residential Buildings Caused by the Northridge Earthquake*, prepared for the United States Department of Housing and Urban Development-Office of Policy Development and Research, NAHB: Upper Marlboro, Maryland.

NIST Special Publication 862, 1994. ICSSCTR 14, Diana Todd et al., Northridge Earthquake.

Popov, E. P., and R.M. Stephen, 1970. *Cyclic Testing of Full Size Steel Connections*, Report No. EERC-70-3, Earthquake Engineering Research Center, University of California, Berkeley, California.

SEAOC, 1994. *Proceedings of the 63rd Annual Convention,* Sacramento, California.

Shepherd, R. and B. Allred, 1995. *Cyclic Testing of Narrow Plywood Shear Walls,* ATC-R-1, Applied Technology Council, 555 Twin Dolphin Dr., Redwood City, California 94605.

SSC 94-06, 1994. *Northridge Building Case Studies*, Eds: Rutherford and Chekene, Engineers. California Seismic Safety Commission, Sacramento, California.

SSC 94-08, 1994. *Compendium Background Report*, California Seismic Safety Commission, Sacramento, California.

Tsai, K.C., and E.P. Popov, 1988. *Steel Beam-Column Joints in Seismic Moment Resisting Frames,* Report No. UCB/EERC-88/19, Earthquake Engineering Research Center, University of California at Berkeley, Berkeley, California.

UCSD, *The Northridge Earthquake of January 17, 1994: Damage Analysis of Selected Freeway Bridges*, 1994. Report No. SSRP-94/06, Structural Systems Research Project, University of California, San Diego.

USGS, 1994. *Accelerograms Recorded at USGS National Strong-Motion Network Stations During the* $M_s=6.6$ *Northridge, California Earthquake of January 17, 1994*, United States Geological Survey Department of the Interior, USGS Open File Report 94-141.

Table AppA-1. *1994 Northridge Earthquake Ground Motion Recorded Near-Fault Rupture [References CDMG, 1994; USGS, 1994; Lindvall et al., 1994*

Recording Station			Site Geology	Distance from Fault (km)		Maximum Acceleration	
Name	Owner (Instrument No.)	Location		Epi-center	Fault Rupture	Comp	PGA (g)
Sepulveda VA Hospital	VA (NSMP)	Ground level		7	10	360°	0.94
						Up	0.48
						270°	0.74
Rinaldi *Receiving Station*	LADWP (5968)	Free field	Alluvium	10	5	318°	0.48
						Up	0.85
						228°	0.84
Arleta *Nordhoff Ave. Fire Station*	CDMG (CSMIP 24087)	Ground level	Alluvium	10	15	90°	0.35
						Up	0.59
						360°	0.29
Jensen Filter Plant *Generator Building*	MWD (NSMP)	Ground level		12	5	022°	0.56
						Up	0.52
						292°	0.98

Table AppA-1. (Continued) *1994 Northridge Earthquake Ground Motion Recorded Near-Fault Rupture [References CDMG, 1994; USGS, 1994; Lindvall et al., 1994*

Recording Station				Distance from Fault (km)		Maximum Acceleration	
Name	**Owner (Instrument No.)**	**Location**	**Site Geology**	**Epi-center**	**Fault Rupture**	**Comp**	**PGA (g)**
Sylmar Converter Station (SCS)	LADWP (306-3)	Free field	Alluvium	12	5	052°	0.61
						Up	0.64
						142°	0.90
Sylmar Converter Station East (SCSE)	LADWP (6273)	Free field	Rock	13	5	015°	0.83
						Up	0.38
						285°	0.49
Sylmar *LA County Hospital*	CDMG (CSMIP 24514)	Free field	Alluvium	16	5	360°	0.91
						Up	0.60
						90°	0.61
Newhall *LA County Fire Station*	CDMG (CSMIP 24279)	Ground level	Alluvium	20	5	90°	0.63
						Up	0.62
						360°	0.61

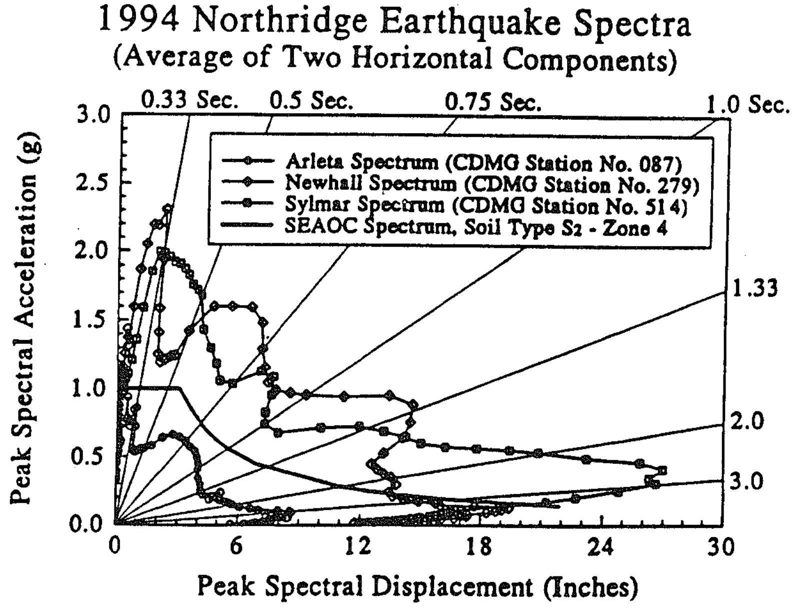


Figure AppA-1. *Comparison of selected spectra from the Northridge earthquake*

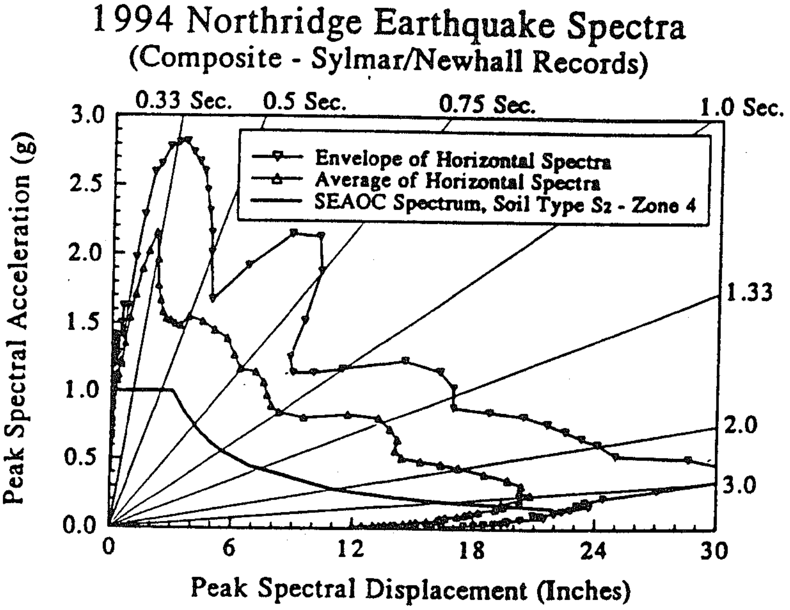
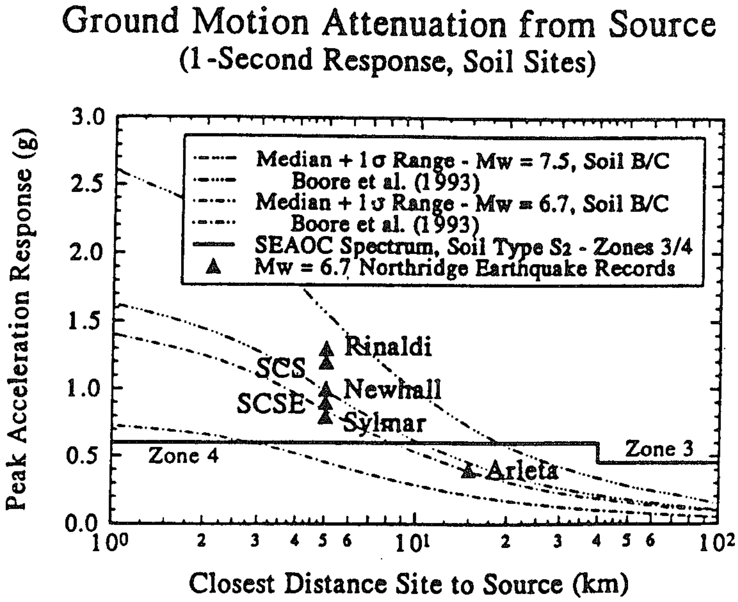


Figure AppA-2. *Composite spectra from the Northridge earthquake*



Ground Motion Attenuation from Source
(1-Second Response, Soil Sites)

Figure AppA-3. *Ground motion attenuation from the Northridge earthquake*

APPENDIX B

Conceptual Framework for Performance-Based Seismic Design

Vision 2000 Committee
Chris Poland (SEAONC), Chair
James Hill (SEAOSC), Vice Chair
Roland Sharpe (SEAONC), Vice Chair
Vitelmo Bertero (SEAONC)
Robert Chittenden (SEAOCC)
Anthony Court (SEAOSD)
Ronald Hamburger (SEAONC)
Joseph Nicoletti (SEAONC)
Richard Philips (SEAOSC)
Scott Stedman (SEAOSD)
Ajit Virdee (SEAOCC)
Nabih Youssef (SEAOSC)

Ad Hoc Members
Vilas Mujumdar (SEAOCC)
Robert Thacker (SEAOCC)
Charles Thiel Jr. (SEAONC)

APPENDIX B

Conceptual Framework for Performance-Based Seismic Design

Introduction

The SEAOC Blue Book has historically targeted a limited set of design objectives, as described in Section 101 of this Commentary. The Recommendations are based largely on application of an equivalent lateral force approach to a single level earthquake design and focus primarily on design of the primary structural system. In recent years, SEAOC has recognized the need and opportunity to develop a new generation of performance-based engineering provisions that embrace a broader scope of design and construction quality assurance issues and that yield more predictable seismic performance over a range of earthquake demands.

SEAOC's Vision 2000 Committee has defined a framework for development of performance-based engineering procedures. These procedures will address a broad range of performance objectives that consider life safety, structural and nonstructural damage control, and maintenance of function over a range of earthquake hazards. These performance-based procedures will embrace new design and analysis approaches that will more directly address the inelastic response of structures and provide alternative procedures to better achieve defined seismic performance objectives.

This Appendix is adapted from SEAOC's Vision 2000 report [OES, 1995] and outlines the conceptual framework developed by Vision 2000. It defines performance-based engineering in terms of performance levels, seismic hazard levels and performance objectives, and provides guidance for the development and incorporation of performance-based engineering guidelines into future editions of the Blue Book.

Overview

The conceptual framework for performance-based engineering is defined as a full range of seismic engineering issues to be addressed in designing structures for predictable and definable seismic performance, within established levels of risk. The methodology is illustrated in a flow chart in Figure AppB-1.

This framework must first be developed into guidelines and then into code provisions before it can be readily applied in building design. Some of the concepts are easily developed; others will require considerable research and trial design applications before they can be developed into a usable form. During the transition from current seismic design practice to performance-based engineering design, it is anticipated that new provisions will be incrementally developed from existing provisions. This will ensure a smooth transition.

This performance-based engineering methodology will apply to all types of structures in all seismic regions. It has been developed in the context of general building design, but may also be applied to bridges and other nonbuilding structures. It is focused primarily on new construction, but also applies, with some limitations, to existing structures. Applications to existing buildings are also being developed in ongoing guideline development projects, including ATC-33 and ATC-40/Proposition 122 [ATC-33; ATC-40].

Performance-based engineering begins with the selection of performance objectives and identification of seismic hazards, continues with conceptual, preliminary and final designs, design verification, and design review, and concludes with quality assurance during construction and building maintenance after construction. Each step is pursued to a greater or lesser extent, depending on the rigor of design required to meet the selected performance objectives. Abbreviated methodologies can be used for simple structures with modest performance objectives.

The “design” and “verification” steps vary depending on the design approach selected and the performance objectives. Building design approaches are outlined in the Vision 2000 report [OES, 1995] and include:

- Comprehensive Design
- Displacement
- Energy
- General Force/Strength
- Simplified Force/Strength
- Prescriptive Approaches

Verification analysis procedures outlined include:

- General Elastic Analysis Procedures
- Component-Based Elastic Analysis Procedure
- Capacity Spectrum Procedure
- Pushover Analysis Methods
- Dynamic Nonlinear Time History Analysis
- Drift Demand Spectrum Analysis

The capacity design philosophy is adopted as an underlying principle of effective seismic design. In capacity design, the inelastic behavior of the structure is controlled by proportioning the structure to yield at predetermined ductile "fuse" locations. Those "fuses" are detailed to accommodate the ductility demand imposed by the earthquake while the rest of the structure remains elastic.

The seismic design process involves multiple sources of uncertainty, including seismic hazard, analysis methods and models, and variability in the construction materials and workmanship. Performance-based engineering techniques must identify and quantify those uncertainties so that the reliability of the design can be estimated and acknowledged.

Each major step in performance-based engineering methodology is described in greater detail in the following subsections. These descriptions are general in nature and are more fully developed in the Vision 2000 report [OES, 1995]. Each is written as a general description of the end product, with some reference to particular applications, and will need to be fully developed.

Selection of Performance Objectives

The first step in performance-based engineering is selection of seismic performance objectives for the design. This selection is made by the client, in consultation with the design professional, based on consideration of the client's expectations, the seismic hazard exposure, economic analysis, and acceptable risk. Performance objectives will typically range from code minimums, usually based on life safety in a rare earthquake, to operational in a very rare earthquake.

A performance objective is a coupling of expected performance level with expected levels of seismic ground motions. A performance level is a damage state, a distinct band in the spectrum of possible seismic damage states as illustrated in Figure AppB-2. Performance levels are defined in terms of damage to the structure and nonstructural components and in terms of consequences to the occupants and functions carried on within the facility. Four performance levels are identified in this Appendix and are described in detail in the Vision 2000 report. These performance levels are as follows:

- **Fully Operational.** Facility continues in operation with negligible damage.
- **Operational.** Facility continues in operation with minor damage and minor disruption in nonessential services.
- **Life Safe.** Life safety is substantially protected, damage is moderate to extensive.
- **Near Collapse.** Life safety is at risk, damage is severe, structural collapse is prevented.

Tables AppB-1 through AppB-6 further define these performance levels in terms of damage to the various components of the building.

The seismic hazard at a given site is represented as a set of earthquake ground motions and associated hazards with specified probabilities of occurrence. Four levels of probabilistic events are proposed as follows:

Levels for Design and Verification

Event	Recurrence Interval	Probability of Exceedence
Frequent	43 years	50% in 30 years
Occasional	72 years	50% in 50 years
Rare	475 years	10% in 50 years
Very rare	970 years	10% in 100 years

Performance objectives are composed of multiple goals; for example, fully operational in the 43-year event, life safe in the 475-year event, and collapse prevention in the 970-year event. For this Vision 2000 report, a set of minimum objectives and enhanced objectives are identified:

- **Minimum Objectives.** The basic objective is defined as the minimum acceptable performance objective for typical new buildings. Essential/hazardous facility and safety critical objectives are defined as minimum objectives for facilities such as hospitals and nuclear material processing facilities, respectively. These three minimum objectives are illustrated in Figure AppB-3 as diagonal lines in the performance objective matrix.
- **Enhanced Objectives.** Other objectives that provide better performance or lower risk than the minimum objectives may be selected at the client's discretion. These objectives are termed enhanced objectives.

The selection of performance objectives sets the acceptance criteria for the design. The performance objectives represent performance levels, or damage levels, expected to result from design ground motions. The performance levels are keyed to limiting values of measurable structural response parameters, such as drift and ductility demand. When the performance objectives are selected, the associated limiting values become the acceptance criteria to be checked in later stages of the design. Limiting values of the response parameters that correlate with the defined performance levels must be established through research. Acceptance criteria are discussed further in Preliminary and Final Design Steps, below.

Site Suitability and Design Ground Motions

Before structural design begins, a site suitability and seismic hazard analysis that considers the proposed performance objectives for the project must be undertaken. The building site should be analyzed for suitability for the selected performance objective, considering any site hazards that may exist. Ground motion design criteria should be established and characterized in a form suitable for the anticipated structural analysis and design methods.

Site suitability analysis includes consideration of the site seismicity, soil type, and potential site hazards. Analysis will typically include determination of soil profile and topography, identification of seismic sources and source mechanisms, and identification of liquefaction potential, tsunami exposure, and other hazards such as flooding and fire hazards from adjacent properties.

Seismic hazard analysis will determine the design ground motion for the specified design events considering all critical seismic sources. Ground motions can be represented as time histories, acceleration response spectra, displacement response spectra, drift demand spectra, or by other means, as required for the design and analysis methods. Response spectra may be elastic or inelastic, as required for the design and analysis. Damping may be accounted for as 2 percent critical for purely elastic response, 5 percent critical for traditional elastic analysis assuming inelastic response, or possibly 10-20 percent critical for inelastic analysis or for specially damped systems. Duration effects and energy content of the expected ground motion need further study before they are directly accounted for in the design.

For simple, regularly configured buildings with basic performance objectives, this step will follow procedures similar to the current code. The site suitability analysis may be based on publicly available maps or commonly known information. The seismic hazard analysis may be nothing more than a seismic zone factor assigned by the building official.

Conceptual Design

Once the performance objectives are selected and the site suitability and seismic ground motions are established, structural design can proceed. Structural design begins with overall conceptual design of the facility including layout, configuration, selection of structural systems and materials, selection of foundation systems and, to varying degrees, selection of nonstructural systems. It is in the conceptual design stage that fundamental decisions are made that determine the ultimate viability of the design.

Modern seismic resistant design relies on the inelastic response of the structure to dissipate much of the input energy generated by major earthquakes. Therefore, it is imperative that the postelastic behavior of the structure be addressed at this stage of design. Capacity design principles should be applied to provide an overview of the inelastic response and to designate the ductile links or "fuses" in the lateral force resisting system. The designated "fuses" will be counted on to yield and to dissipate the input energy of the earthquake while the rest of the system remains elastic. The conceptual overview obtained provides the design team with a clear conceptual understanding of the intended inelastic response of the structure and allows the design and quality assurance programs to focus on the critical links in the system. At the final design and detailing stage, these critical links must be rigorously detailed to provide the ductility required.

In making fundamental design decisions at the conceptual design stage, the following guidelines to good seismic design should be considered:

1. Use simple, symmetrical and regular configurations where possible.
2. Use lighter building weight rather than heavier.
3. Avoid high slenderness ratios.
4. Provide redundancy and ductility to overcome seismic design uncertainties.
5. Provide sufficient stiffness to limit drift-related damage.
6. Provide sufficient flexibility to limit acceleration-related damage.
7. Provide toughness (energy dissipation capacity) and stability in strength and stiffness in postelastic cyclic behavior.
8. Provide uniform and continuous distribution of strength, stiffness, ductility, and toughness.
9. Provide structural strength and stiffness compatible with the foundations and soils type.
10. Use relatively short spans and close column spacings.
11. Tie vertical elements together at each level, including the foundation.
12. Use capacity design principles to control the inelastic behavior by identifying and providing a system of ductile links to absorb the inelastic response.
13. Consider use of energy dissipation devices as a design strategy.
14. Consider use of seismic isolation devices as a design strategy.

Since conceptual design is closely tied to the ultimate performance of a building, guidelines specifying appropriate limitations on the materials, configuration, and structural system are needed for each performance objective. These must be defined in terms that are usable at the conceptual design stage. The level of restriction should increase in severity with the increase in performance objective and should reflect the excellent historic performance of regularly configured structural systems composed of well-detailed ductile materials.

Preliminary and Final Design Steps

With the performance objective selected, the ground motion for critical earthquakes established, and the conceptual design resolved, preliminary and final designs can proceed. Design procedures will differ depending on the performance objectives and the design and analysis approaches taken, however the basic steps and issues considered are similar. This Vision 2000 report outlines several of the possible design and verification approaches.

A major challenge to performance-based engineering is to develop simple yet effective methods for designing, analyzing, and verifying the design of structures so that they reliably meet the selected performance objectives. The possible approaches to the design problem include familiar force/strength approaches, displacement approaches, energy approaches and prescriptive design approaches. Verification methods include various elastic and inelastic analysis procedures.

The preliminary and final design stages involve sizing and detailing the structural framing system and the nonstructural systems so that the performance objectives can be met. Performance objectives must be translated into engineering to establish an acceptance criteria, defined as limiting values in the response parameters that become targets for the design. These criteria include drift and deformation limits, acceleration and force limits, yield limits, ductility limits and energy dissipation levels that must be met in order for the structural response to be consistent with the performance objectives. Those acceptance criteria are discussed further in Verification at Each Design Step, below.

In preliminary design, structural framing elements are sized and checked against selected criteria. This involves designing to meet at least two criteria, one for operational continuity in a frequent earthquake and one for life safety or collapse prevention in a rarer earthquake. As part of preliminary element sizing, elements should be proportioned so that the expected postelastic yield patterns will be consistent with the capacity-based conceptual design.

In final design and detailing of the structure, all selected performance goals must be considered. As appropriate, both the elastic and inelastic responses should be considered and compared to the limiting response parameters.

For capacity designed structures, the ductile link regions should be detailed to ensure acceptable inelastic behavior. The details must provide suitable ductility to extend the structural response from the elastic limit to the inelastic limit imposed by the design ground motions. The rest of the structure will be designed to remain elastic while the ductile links develop their inelastic capacity.

Nonstructural elements and attachments should be designed to accommodate the elastic and inelastic deformations of the structure. Nonstructural elements may be designed to be isolated from the structural system, or to be integral parts and participate in the building's inelastic response. In either case, these elements must be designed and checked to meet the performance objectives.

Verification at Each Design Step

At each stage of design, an analysis is performed to verify that the selected performance objectives are being met. The specific extent and methodology of the verification will vary with the performance objectives and the design approach used.

Structural response, as measured by certain quantifiable parameters, must be consistent with the performance objectives and associated acceptance criteria. The acceptance criteria consist of limiting values in structural response parameters, associated with selected performance levels or damage levels for specified levels of ground motion. Within a particular building, the design of specific components may be controlled by the same or different response parameters for the same or different performance objectives. Typical response parameters verified may include:

- Stress ratios
- Drift and deformation ratios
- Structural accelerations
- Ductility demand ratios
- Energy dissipation demand vs. capacity

Typical limiting values for these response parameters must be established for each performance level through research that includes laboratory testing of specific components, as well as calibrating the limiting values by analyzing buildings that have experienced measurable damage in past earthquakes and for which strong motion records are available. Verification will include both the structural and the nonstructural components. In many cases, nonstructural components can be prequalified by being prescriptively designed to meet the target parameters of the structural design.

For certain simple or prescriptive design approaches, no formal verification procedures other than plan review by the building official may be performed. In such cases, the basic design requirements will need to be sufficiently conservative to reasonably ensure that the intended basic performance objective will be achieved.

Verification procedures will involve both elastic and inelastic analysis methods. Elastic analysis procedures for checking stress ratios and drift are currently well known and widely used. Several simplified inelastic approaches, such as the pushover analysis, are outlined in the Vision 2000 report [OES, 1995], as well as in ATC-33 and ATC40 [ATC-33, ATC-40].

Design Review

An important quality assurance step in the design process is competent independent design review. This design review includes both independent peer review and plan review by the building official.

Schools and hospitals in California are designed to standards very similar to those used for other buildings in California, yet they tend to perform much better in earthquakes, largely because of the rigor of the plan review during design and the quality assurance provided during construction.

For simpler buildings and for prescriptive designs, the building official will provide the only independent review of the design. It is very important, therefore, that the building official understand the intent of the design and be able to provide a thorough and competent review of the plans.

For all other buildings, including specialized, irregular, or important structures, independent peer review should be required. Such review should be undertaken at the end of the conceptual design and final design stages in order to provide independent professional assessment of the design, the assumptions, modeling, analysis, and effectiveness of the design in meeting the performance goals.

Quality Assurance During Construction

Regardless of the quality of the design and sophistication of the analysis, performance-based engineering will not be successful without adequate quality assurance during construction.

The quality assurance process involves a team of players, including the design professional, peer reviewer, building official, special inspectors, testing agencies, and the contractor. The design professional must be involved to ensure that the design intent is properly interpreted and that the critical elements in the building system are recognized, properly constructed, and properly inspected and tested. The building official is legally involved in all projects and must verify that the project is constructed in accordance with building codes and that the specified inspection, testing, and structural observation are completed.

The special inspector must, at the direction of the design professional, inspect the critical elements of the system and must ensure that those elements are constructed in strict compliance with the plans. The testing agency must verify the quality of construction materials and the qualifications of the special inspector.

The scope of the quality assurance program may be limited in simple structures with modest performance objectives or in prescriptively designed structures. The quality assurance program must be rigorous for complex structures with demanding performance objectives. In each case, the quality assurance program should be defined by the structural engineer of record.

Building Maintenance and Function

Performance-based engineering does not end with the completion of construction. The responsibilities merely shift. The building condition, configuration, and use will evolve over the life of the structure. The owner and the building official must remain vigilant to ensure that the earthquake resistant system is not adversely altered through future remodels and renovations. Further, the owner must ensure that the structure is maintained, that elements are not allowed to corrode or deteriorate, and that change of use does not result in higher static live loads such as storage inventory that might significantly change the mass and thereby the dynamic response of the building without proper analysis and validation.

Maintenance requirements will vary with the type of construction and the selected performance objectives. Seismic designs that use sophisticated technologies such as active or passive energy dissipation devices or relatively degradable materials must be properly maintained on a regular basis to perform as expected. Prescriptive designs using traditional building materials may have lower maintenance requirements.

Uncertainty, Risk, and Reliability

Performance-based engineering yields structures with predictable performance within defined levels of risk and reliability. There are multiple sources of uncertainty in the seismic design process. There is uncertainty regarding the seismic ground motions, the design and analysis techniques and modeling assumptions, the variability in the materials, workmanship and construction quality, and the changes to the structure over its life due to material deterioration, wear, and structural or nonstructural modifications.

It is an important task of performance-based engineering to recognize, identify, and quantify those uncertainties so that the levels of reliability and risk can be established and acknowledged by the design professional, the client, the legal profession and the public.

Performance-based engineering will, of necessity, start with a set of seismic design guidelines, incrementally improved over present code, and with modest claims of reliability in performance prediction. As new design and analysis techniques are developed and can be proven to accurately predict the seismic performance observed in recorded earthquakes, design guidelines will be modified and reliability will be progressively improved.

Research Needs

The development of this conceptual framework into a guideline for performance-based engineering will require considerable new basic research, applied research, calibration studies, trial designs, and actual guideline preparation. Each facet must be embraced and supported in a balanced manner if the goal is to be achieved.

In general, research related to this conceptual framework is needed in the following general areas:

1. Validate the appropriateness of the four performance levels and the specific parameters used to define their minimum performance states.
2. Validate the appropriateness of the four seismic hazard definitions through consideration of the social, economic, and political impact seismic design and seismic performance has on communities in various seismic regions.
3. Establish appropriate acceptance criteria for site performance in terms of permissible settlement, lateral spreading, liquefaction, faulting, etc. for each performance objective.
4. Identify appropriate ground motion parameters needed for the possible design approaches and analysis checks that yield the best and most appropriate characterization of the seismic hazard.
5. Develop guidelines for conceptual design that address all facets of performance-based engineering from the project start.

6. Identify and develop appropriate design approaches and verification methods that are fully compatible with the entire structural/earthquake engineering design effort.
7. Identify the key characteristic of a proper design review based on current and successful efforts and develop appropriate guidelines. Determine if there is a need to augment and/or change the basic functions of the traditional building department as they relate to achieving the various performance objectives.
8. Based on the performance of buildings in past earthquakes, determine the significance that construction quality has on performance. Establish guidelines for appropriate construction quality assurance programs for buildings built to each of the performance objectives.
9. Develop code provisions that will provide for the maintenance of a building's structural system for the life of the building.
10. Develop techniques for systematically defining the levels of risk and reliability inherent in the guidelines and provisions written for performance-based engineering. Develop an acceptance criteria for each and monitor the performance of buildings designed under the guidelines to ensure that they are yielding structures of predictable performance within defined levels of risk and reliability.

These research recommendations are somewhat general and provide only an indication of the level of research and development that is needed in each of the key areas of performance-based engineering. In each case, it is important that sufficient work is done to verify the accuracy of the techniques purposed, their compatibility with the overall conceptual framework as defined, and their consistency with the historic performance of buildings in past earthquake

References

OES, 1995. California Office of Emergency Services, *Vision 2000: Performance Based Seismic Engineering of Buildings*. Prepared by Structural Engineers Association of California, Sacramento, California.

ATC-33, in progress. *Guidelines for the Seismic Rehabilitation of Buildings,* Applied Technology Council, Redwood City, California.

ATC-40, 1996. *Recommended Methodology for the Seismic Evaluation and Retrofit of Existing Concrete Buildings,* Applied Technology Council, Redwood City, California.

Table AppB-1. *General Damage Descriptions by Performance Levels and Systems*

	Performance Level				
System Description	**Fully 10 Operational 9**	**8 Operational 7**	**6 Life Safety 5**	**4 Near Collapse 3**	**2 Collapse 1**
Overall Building Damage	Negligible	Light	Moderate	Severe	Complete
Permissible Transient Drift	< 0.2% ±	< 0.5% ±	< 1.5% ±	< 2.5% ±	> 2.5% ±
Permissible Permanent Drift	Negligible	Negligible	< 0.5% ±	< 2.5% ±	> 2.5% ±
Vertical Load-Carrying Element Damage	Negligible	Negligible	Light to moderate, but substantial capacity remains to carry gravity loads	Moderate to heavy, but elements continue to support gravity loads	Partial to total loss of gravity load support
Lateral Load-Carrying Element Damage	Negligible. Generally elastic response; no significant loss of strength or stiffness	Light. Nearly elastic response; original strength and stiffness substantially retained; minor cracking/ yielding of structural elements; repair implemented at convenience	Moderate. Reduced residual strength and stiffness, but lateral system remains functional	Negligible residual strength and stiffness; no story collapse mechanisms, but large permanent drifts; secondary structural elements may completely fail	Partial or total collapse; primary elements may require demolition
Damage to Architectural Systems	Negligible damage to cladding, glazing, partitions, ceilings, finishes, etc.; isolated elements may require repair at users convenience	Light to moderate damage to architectural systems; essential and select protected items undamaged; hazardous materials contained	Moderate to severe damage to architectural systems, but large falling hazards not created; major spills of hazardous materials contained	Severe damage to architectural systems; some elements may dislodge and fall	Highly dangerous falling hazards; destruction of components

Table AppB-1. (Continued) *General Damage Descriptions by Performance Levels and Systems*

	Performance Level				
System Description	**Fully 10 Operational 9**	**8 Operational 7**	**6 Life Safety 5**	**4 Near Collapse 3**	**2 Collapse 1**
Egress Systems	Not impaired	No major obstructions in exit corridors; elevators can be restarted perhaps following minor servicing	No major obstructions in exit corridors; elevators may be out of service for extended period	Egress may be obstructed	Egress may be highly or completely obstructed
Mechanical/ Electrical/ Plumbing/utility Services	Functional	Equipment essential to function and fire/life safety systems operate; other systems may require repair; temporary utility service provided as required	Some equipment dislodged or overturned; many systems not functional; piping conduit ruptured	Severe damage and permanent disruption of systems	Partial or total destruction of systems; permanent disruption of systems
Damage to Contents	Some light damage to contents may occur; hazardous materials secured and undamaged	Light to moderate damage; critical contents and hazardous materials secured	Moderate to severe damage to contents; major spills of hazardous materials contained	Severe damage to contents; hazardous materials may not be contained	Partial or total loss of contents
Repair	Not required	At owner/tenants convenience	Possible; building may be closed	Probably not practical	Not possible
Effect on Occupancy	No effect	Continuous occupancy possible	Short term to indefinite loss of use	Potential permanent loss of use	Permanent loss of use

Table AppB-2. *Performance Levels and Permissible Structural Damage — Vertical Elements*

		Performance Level			
Elements	**Type**	**10 Fully Operational 9**	**8 Operational 7**	**6 Life Safety 5**	**4 Near Collapse 3**
Concrete Frames	Primary	Negligible	Minor hairline cracking (0.02"); limited yielding possible at a few locations; no crushing (strains below 0.003)	Extensive damage to beams; spalling of cover and shear cracking (< 1/8") for ductile columns; minor spalling in nonductile columns; joints cracked < 1/8" width	Extensive cracking and hinge formation in ductile elements; limited cracking and/or splice failure in some nonductile columns; severe damage in short columns
	Secondary	Negligible	Same as primary	Extensive cracking and hinge formation in ductile elements; limited cracking and/or splice failure in some nonductile columns; severe damage in short columns	Extensive spalling in columns (possible shortening) and beams; severe joint damage; some reinforcing buckled
SteelMoment Frames	Primary	Negligible	Minor local yielding at a few places; no observable fractures; minor buckling or observable permanent distortion of members	Hinges form; local buckling of some beam elements; severe joint distortion; isolated connection failures; a few elements may experience fracture	Extensive distortion of beams and column panels; many fractures at connections
	Secondary	Negligible	Minor local yielding at a few places; no fractures; minor buckling or observable permanent distortion of members	Extensive distortion of beams and column panels; many fractures at connections	Extensive distortion of beams and column panels; many fractures at connections

Table AppB-2. (Continued) *Performance Levels and Permissible Structural Damage — Vertical Elements*

		Performance Level			
Elements	**Type**	**10 Fully Operational 9**	**8 Operational 7**	**6 Life Safety 5**	**4 Near Collapse 3**
Braced Steel Frames	Primary	Negligible	Minor yielding or buckling of braces; no out-of-plane distortions	Many braces yield or buckle but do not totally fail; many connections may fail	Extensive yielding and buckling of braces; many braces and their connections may fail
	Secondary	Negligible	Same as primary	Same as primary	Same as primary
Concrete Shear Walls	Primary	Negligible	Minor hairline cracking (0.02") of walls; coupling beams experience cracking < 1/8" width	Some boundary elements distress including limited bar buckling; some sliding at joints; damage around openings; some crushing and flexural cracking; coupling beams—extensive shear and flexural cracks; some crushing, but concrete generally remains in place	Major flexural and shear cracks and voids; sliding at joints; extensive crushing and buckling of rebar; failure around openings; severe boundary element damage; coupling beams shattered, virtually disintegrated
	Secondary	Negligible	Minor hairline cracking of walls, some evidence of sliding at construction joints; coupling beam experience cracks < 1/8" width, minor spalling	Major flexural and shear cracks; sliding at joints; extensive crushing; failure around openings; severe boundary element damage; coupling beams shattered, virtually disintegrated	Panels shattered, virtually disintegrated

Table AppB-2. (Continued) *Performance Levels and Permissible Structural Damage — Vertical Elements*

		Performance Level			
Elements	**Type**	**10 Fully Operational 9**	**8 Operational 7**	**6 Life Safety 5**	**4 Near Collapse 3**
Unreinforced Masonry Infill Walls	Primary	Negligible	Minor cracking (<1/8") of masonry infills and veneers; Minor spalling in veneers at a few corner openings	Extensive cracking and some crushing, but wall remains in place; no falling units; extensive crushing and spalling of veneers at corners of openings	Extensive cracking and crushing; portions of face course shed
	Secondary	Negligible	Same as primary	Same as primary	Extensive cracking and crushing; portions of face course shed
URM Bearing Walls	Primary	Negligible	Minor cracking (< 1/8") of masonry infills and veneers; minor spalling in veneers at a few corner openings; no observable out-of-plane offsets	Extensive cracking; noticeable in-plane offsets of masonry and minor out-of-plane offsets	Extensive cracking; face course and veneer may peel off; noticeable in-plane and out-of-plane offsets
	Secondary	Negligible	Same as primary	Same as primary	Same as primary
Reinforced Masonry Walls	Primary	Negligible	Minor cracking (< 1/8"); no out-of-plane offsets	Extensive cracking (< 1/4") distributed throughout wall; some isolated crushing	Crushing; extensive cracking; damage around openings and at corners; some fallen units
	Secondary	Negligible	Same as primary	Crushing; extensive cracking; damage around openings and at corners; some fallen units	Panels shattered, virtually disintegrated

Table AppB-2. (Continued) *Performance Levels and Permissible Structural Damage — Vertical Elements*

	Performance Level				
Elements	**Type**	**10 Fully Operational 9**	**8 Operational 7**	**6 Life Safety 5**	**4 Near Collapse 3**
Wood Stud Walls	Primary	Negligible	Distributed minor hairline cracking of gypsum and plaster veneers	Moderate loosening of connections and minor splitting of members	Connections loose, nails partially withdrawn; some splitting of members and panel; veneers shear off
	Secondary	Negligible	Same as primary	Connections loose, nails partially withdrawn; some splitting of members and panel	Sheathing shears off; let-in braces fracture and buckle; framing split and fractured
Foundations	General	Negligible	Minor settlement and negligible tilting	Total settlements <6" and differential settlements <1/2" in 30 feet	Major settlements and tilting

Table AppB-3. *Performance Levels and Permissible Structural Damage — Horizontal Elements*

	Performance Level			
Elements	10 **Fully Operational** 9	8 **Operational** 7	6 **Life Safety** 5	4 **Near Collapse** 3
Metal Deck Diaphragms	Negligible	Connections between deck units and from deck to framing intact; minor distortions	Some localized failure of welded connections of deck to framing and between panels; minor local buckling of deck	Large distortion with buckling of some units and tearing of many welds and seam attachments
Wood Diaphragms	Negligible	No observable loosening, withdrawal of fasteners, or splitting of sheathing or framing	Some splitting at connections, loosening of sheathing; observable withdrawal of fasteners; splitting of framing and sheathing	Large permanent distortion with partial withdrawal of nails and splitting of elements
Concrete Diaphragms	Negligible	Distributed hairline cracking and a few minor cracks (< 1/8") or larger size	Extensive cracking (<1/4") and local crushing and spalling	Extensive crushing and observable offset across many cracks

Table AppB-4. *Performance Levels and Permissible Damage — Architectural Elements*

Element	Performance Level			
	10 Fully Operational 9	8 Operational 7	6 Life Safety 5	4 Near Collapse 3
Cladding	Negligible damage	Connections yield; some cracks or bending in cladding	Severe distortion in connections; distributed cracking, bending, crushing and spalling of cladding elements; some fracturing of cladding, falling of panels prevented	Severe damage to connections and cladding; some falling of panels
Glazing	Generally no damage; isolated cracking possible	Some broken glass; falling hazards avoided	Extensive broken glass; some falling hazards	General shattered glass and distorted frames; widespread falling hazards
Partitions	Negligible damage; some hairline cracks at openings	Cracking to about 1/16" at openings; crushing and cracking at corners	Distributed damage; some severe cracking; crushing and wracking in some areas	Severe wracking and damage in many areas
Ceilings	Generally negligible damage; isolated suspended panel dislocations or cracks in hard ceilings	Minor damage; some suspended ceilings disrupted, panels dropped; minor cracking in hard ceilings	Extensive damage; dropped suspended ceilings; distributed cracking in hard ceilings	Most ceilings damaged; most suspended ceilings dropped; severe cracking in hard ceilings
Light Fixtures	Negligible damage; pendant fixtures sway	Minor damage; some pendant lights broken; falling hazards prevented	Many broken light fixtures; falling hazards generally avoided in heavier fixtures (> 20 lbs.±)	Extensive damage; falling hazards occur
Doors	Negligible damage	Minor damage	Distributed damage; some racked and jammed doors	Distributed damage; many racked and jammed doors
Elevators	Elevators operational with isolated exceptions	Elevators generally operational; most can be restarted	Some elevators out of service	Many elevators out of service

Table AppB-5. *Performance Levels and Permissible Damage — Mechanical/Electrical/Plumbing Systems*

Element	**Performance Level**			
	10 Fully Operational 9	**8 Operational 7**	**6 Life Safety 5**	**4 Near Collapse 3**
Mechanical Equipment	Negligible damage; all remain in service	Minor damage; some units not essential to function out-of-service	Many units nonoperational; some slide or overturn	Most units nonoperational; many slide or overturn; some pendant units fall
Ducts	Negligible damage	Minor damage, but systems remain in service	Some ducts rupture; some supports fail, but ducts do not fall	Most systems out of commission; some ducts fall
Piping	Negligible damage	Minor damage; minor leaking may occur	Some pipes rupture at connections; many supports fail; few fire sprinkler heads fail	Many pipes rupture; supports fail; some piping systems collapse
Fire Alarm Systems	Functional	Functional	Not functional	Not functional
Emergency Lighting Systems	Functional	Functional	Functional	Not functional
Electrical Equipment	Negligible damage	Minor damage; panels restrained; isolated loss of function in secondary systems	Moderate damage; panels restrained from overturning; some loss of function and service in primary systems	Extensive damage and loss of service

Table AppB-6. *Performance Levels and Permissible Damage — Contents*

Element	**Performance Level**			
	10 Fully Functional 9	**8 Operational 7**	**6 Life Safety 5**	**4 Near Collapse 3**
Furniture	Negligible effects	Minor damage; some sliding and overturning	Extensive damage from sliding, overturning, leaks, falling debris, etc.	Extensive damage from sliding, overturning, leaks, falling debris, etc.
Office Equipment	Negligible effects	Minor damage; some sliding and overturning	Extensive damage from sliding, overturning, leaks, falling debris, etc.	Extensive damage from sliding, overturning, leaks, falling debris, etc.
Computer Systems	Operational	Minor damage; some sliding and overturning; mostly functional	Extensive damage from sliding, overturning, leaks, falling debris, etc.	Extensive damage from sliding, overturning, leaks, falling debris, etc.
File Cabinets	Negligible damage	Minor damage; some sliding and overturning	Extensive damage from sliding, overturning, leaks, falling debris, etc.	Extensive damage from sliding, overturning, leaks, falling debris, etc.
Bookshelves	Negligible damage	Minor damage; some overturning and spilling	Extensive damage from leaks, falling debris, overturning, etc.	Extensive damage from leaks, falling debris, overturning, etc.
Storage Racks And Cabinets	Negligible damage; overturning restrained	Minor damage; overturning restrained; some spilling	Extensive damage from leaks, falling debris, overturning, spilling, etc.	Extensive damage from leaks, falling debris, overturning, spilling, etc.
Art Works, Collections	Minor damage; overturning restrained	Moderate damage; overturning restrained, some falling	Extensive damage from leaks, falling debris, overturning, spilling, etc.	Extensive damage from leaks, falling debris, overturning, spilling, etc.
Hazardous Materials	Negligible damage; overturning and spillage restrained	Negligible damage; overturning and spillage restrained	Minor damage; overturning and spillage generally restrained	Severe damage; some hazardous materials released

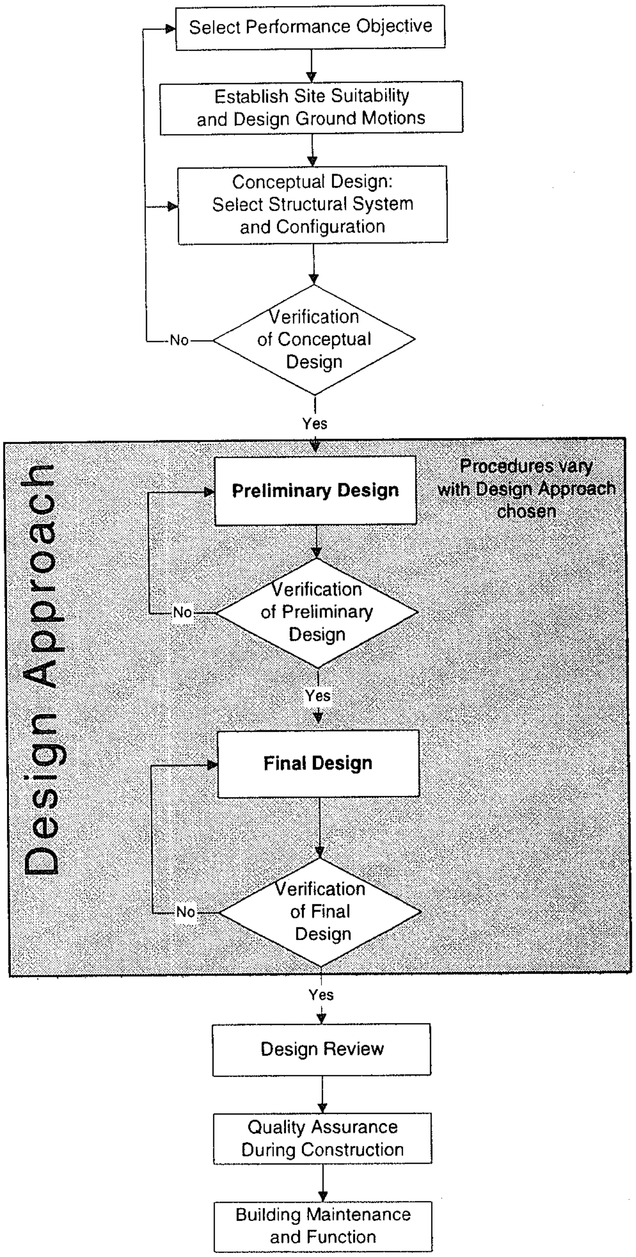
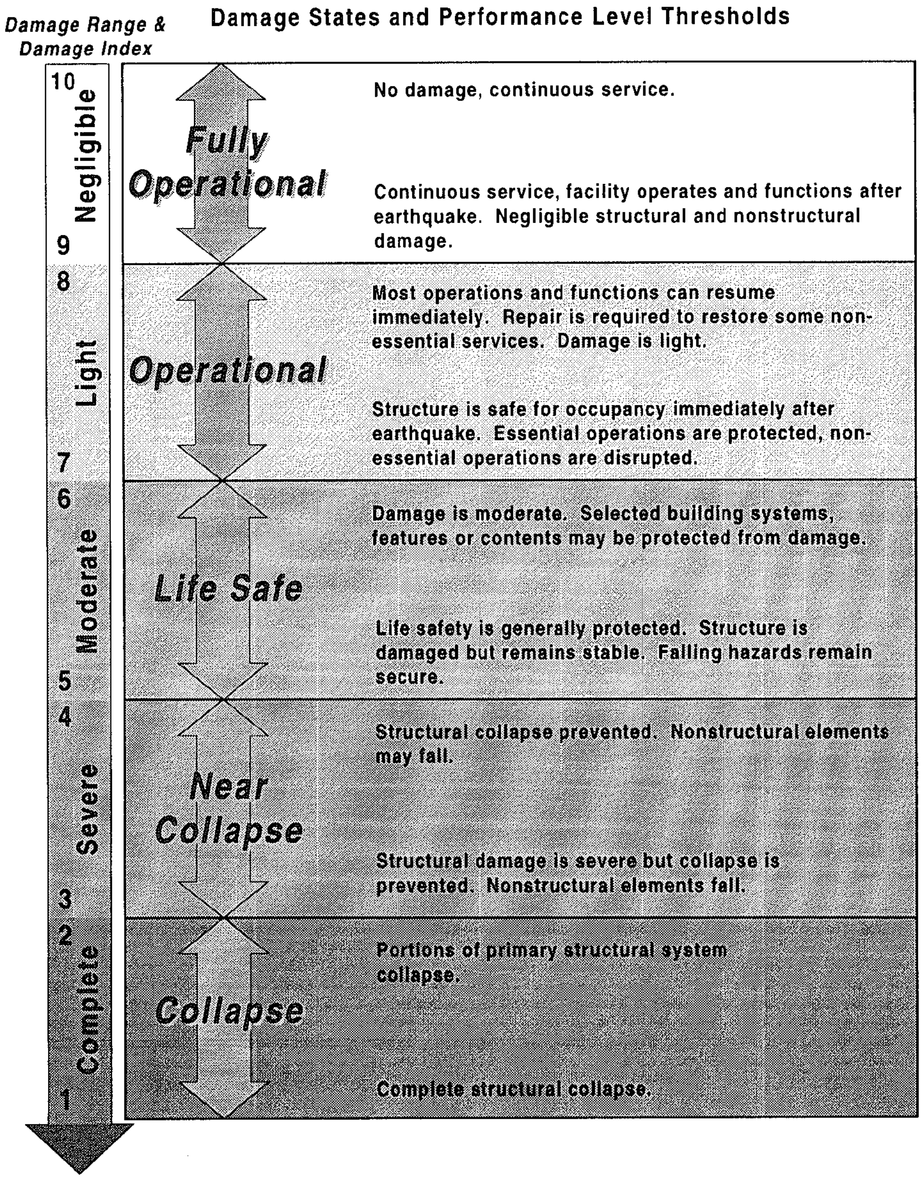

Select Performance Objective
Establish Site Suitability and Design Ground Motions
Conceptual Design: Select Structural System and Configuration
Verification of Conceptual Design
No
Yes
Design Approach
Procedures vary with Design Approach chosen
Preliminary Design
Verification of Preliminary Design
No
Yes
Final Design
Verification of Final Design
No
Yes
Design Review
Quality Assurance During Construction
Building Maintenance and Function

Figure AppB-1. Methodology for performance-based engineering



Damage Range & Damage Index		Damage States and Performance Level Thresholds
10 Negligible 9	Fully Operational	No damage, continuous service. Continuous service, facility operates and functions after earthquake. Negligible structural and nonstructural damage.
8 Light 7	Operational	Most operations and functions can resume immediately. Repair is required to restore some non-essential services. Damage is light. Structure is safe for occupancy immediately after earthquake. Essential operations are protected, non-essential operations are disrupted.
6 Moderate 5	Life Safe	Damage is moderate. Selected building systems, features or contents may be protected from damage. Life safety is generally protected. Structure is damaged but remains stable. Falling hazards remain secure.
4 Severe 3	Near Collapse	Structural collapse prevented. Nonstructural elements may fall. Structural damage is severe but collapse is prevented. Nonstructural elements fall.
2 Complete 1	Collapse	Portions of primary structural system collapse. Complete structural collapse.

Figure AppB-2. *Spectrum of seismic damage states*

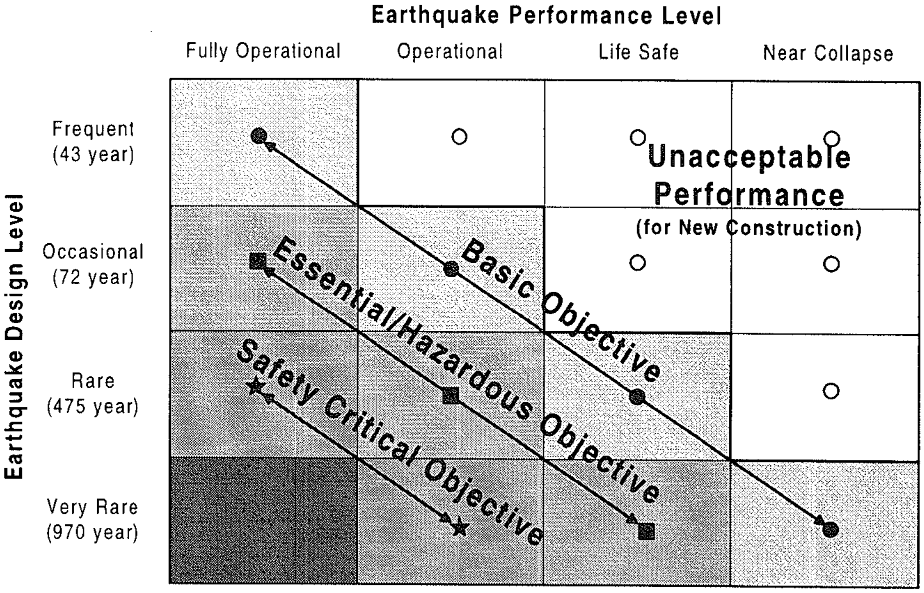


Figure AppB-3. Recommended seismic performance objectives for buildings

APPENDIX C
SEAOC Strength Design Code Change Proposal

The following Seismology Committee members contributed to the development of these recommendations:

Robert Bachman (SEAOSC), Chair
Stacy Bartoletti
Kalman Benuska (SEAOSC)
Vera Berger (SEAOSD)
David Bonneville (SEAONC)
Dominic Campi (SEAONC)
Robert Chittenden (SEAOCC)
Neville Donovan (SEAOSC)
Susan Dowty (SEAOSC)
Tom Hale (SEAOCC)
Ronald Hamburger (SEAONC)
Edward Hanlon (SEAOSD)
Gary Hart (SEAOSC)
Saif Hussain (SEAOSC)
Mark Jokerst (SEAONC)
William Joyner (SEAONC)
Charles Kircher (SEAONC)
Marshall Lew (SEAOSC)
Douglas Magee (SEAOSD)
Dan McNaughton (SEAOSD)
Roumen Mladjov (SEAONC)
Lalliana Mualchin (SEAOCC)
Vilas Mujumdar (SEAOCC
Richard Phillips (SEAOSC)
Maury Power (SEAONC)
John Ragsdale (SEAOCC)
Thomas Sabol (SEAOSC)
Ali Sadre (SEAOSD)
Carl Schulze (SEAOSD)
Robert Thacker (SEAOCC)
Ajit Virdee (SEAOCC)
Nabih Youssef (SEAOSC)
E.M. Wang (SEAOSD)
Ted Zsutty (SEAONC)

APPENDIX C

SEAOC Strength Design Code Change Proposal

Introduction

The SEAOC Seismology Committee has developed this appendix to serve as a preview of the proposed seismic design requirements of the next edition of the Blue Book. It represents an advancement of the current Blue Book in a number of important areas and has been submitted as a Code Change Proposal for the 1997 UBC.

Note as to format: Appendix C has been presented in the numbering proposed for 1997 UBC and in the format of the ICBO Code Change Proposal (sections unchanged from the 1994 UBC not repeated), therefore it should be used in conjunction with the 1994 UBC. The new numbering format is generally offset from the numbering system found in the 1994 UBC by twenty-seven sections (e.g., Section 1630 in 1994 UBC is Section 1647 in the proposed 1997 UBC numbering format).

The Seismology Committee has developed this proposal over a three year period in coordination with a parallel effort by the Building Seismic Safety Council (BSSC) for the 1997 NEHRP Provisions. The NEHRP Provisions serve as the source document for other US model building codes (BOCA and SBC). Therefore, this proposal represents an important SEAOC contribution to the cooperative development of a national code by the year 2000.

The key concepts contained in this proposal are as follows:

1. The adoption of ASCE-7 load factors for strength-based load combinations. In addition, working stress load combinations are maintained as an alternate.
2. The incorporation of a Redundancy/Reliability Factor (ρ), which is intended to encourage redundant lateral force resisting systems by penalizing nonredundant ones through higher lateral force requirements.
3. The incorporation of near-source factors (N_a or N_v) in Seismic Zone 4 which is intended to recognize the amplified ground motions which occur at close distances to the fault. Two factors are used to represent separately the lesser amplification in the short period range and the greater in the long period range.

4. The adoption of a new set of soil profile categories (from 1994 NEHRP) which are used in combination with Seismic Zone Factors (Z), and near-source factors to provide site-dependent ground motion coefficients (C_a and C_v) which define ground motion response within the acceleration and velocity-controlled ranges of the spectrum. The equivalent lateral force equation differs from the current one in two ways: the constant velocity portion is defined now by $1/T$, as opposed to $1/T^{2/3}$, causing it to drop more rapidly in that range, and the plateau in the constant acceleration domain varies with C_a rather than being a constant value for all soil profiles.
5. Substantial revisions to lateral force requirements for elements of structures, nonstructural components and equipment supported by structures. These provisions more accurately represent lateral forces by recognizing varying diaphragm accelerations, component amplification, component response modification, and ground motion response. Similar changes are proposed for nonbuilding structures.
6. A simplified design base shear calculation which is permitted for one and two-story dwellings, one to three-story light frame construction and other one and two-story buildings as permitted.
7. The R-factor has been broken into two components, R_o and R_d representing overstrength and system ductility. However, it was found that the requirements for defining the plastic mechanism analysis required for the R_o calculation could not be codified in simple language while guaranteeing accuracy, so at present this parameter is simply listed in a table along with R_d.

The use of near field factors (N_a or N_v) represents the most significant difference between this proposal and the developing 1997 NEHRP Provisions which will address near field effect through the use of spectral values maps which were developed by USGS. The maps represent a major research effort but were not completed (for design application) in time for use in this proposal.

An important concept in the proposal is the use of elastic response parameters to define unreduced forces and displacements ($R = 1$) for calculations involving drift and deformation compatibility and in dynamic analysis. In addition, the parameter E_M has been introduced to represent the maximum earthquake force that can be developed in the structure for use in addressing nonductile conditions, similar to the $3/8R_w$ parameter in the current code. E_m is used to define collector strength requirements.

In summary, the proposal incorporates a number of important lessons from recent earthquakes and recent advances from other sources. In general it is intended to provide parity with current requirements, except in near field locations, softer soil sites in lower seismic zones and for systems with poor redundancy.

An Addendum on pages 503-504 outlines minor changes made by ICBO when these code changes were adopted at its annual meeting on September 10, 1996.

APPENDIX C

SEAOC Strength Design Code Change Proposal

Requirements

The following code change provisions are part of a reorganization of Chapter 16 of the *Uniform Building Code.* These code changes are being proposed in conjunction with other code changes not presented here, and represent new proposed section numbering. Underline and strikeout indicate proposed additions and deletions. Underline and strikeout of section and formula numbering has been omitted for clarity.

Modify: **UBC §1603.0** **Notations.** *Add section as follows and redesignate remaining section accordingly:*

D = dead load.

E = earthquake load set forth in Section UBC §1645.1.1.

E_m = Estimated maximum earthquake force that can be developed in the structure as set forth in §UBC §1645.1.1.

H = loads due to lateral pressure of soil and water in soil.

F = load due to fluids.

L = live load, except roof live load, including any permitted live load reduction.

L_r = roof live load including any permitted live load reduction.

P = ponding load.

S = snow load.

T = self straining force and effects arising from contraction or expansion resulting from temperature change, shrinkage, moisture change, creep in component materials, movement due to differential settlement, or combinations thereof.

W = wind load.

Add new section: UBC §1612 Combinations of Loads

UBC §1612.1 General. Buildings and other structures and all portions thereof shall be designed to resist the load combinations specified in Sections UBC §1612.2 or UBC §1612.3. and, where required by Chapter 16, Division III, or Chapters 18 through 23, the special seismic load combinations of §UBC §1612.4.

The most critical effect may occur when one or more of the contributing loads are not acting. All applicable loads shall be considered, including both earthquake and wind, in accordance with the specified load combinations.

UBC §1612.2 Load Combinations Using Strength Design or Load and Resistance Factor Design

UBC §1612.2.1 Basic Load Combinations. Where load and resistance factor design (strength design) is used, structures and all portions thereof shall resist the most critical effects from the following combinations of factored loads:

$$1.4D \tag{12-1}$$

$$1.2D + 1.6L + 0.5\,(L_r \text{ or } S) \tag{12-2}$$

$$1.2D + 1.6\,(L_r \text{ or } S) + (f_1L \text{ or } 0.8W) \tag{12-3}$$

$$1.2\mathrm{D} + 1.3W + f_1L + 0.5\,(L_r \text{ or } S) \tag{12-4}$$

$$1.2D + 1.0E + (f_1L + f_2S) \tag{12-5}$$

$$0.9D \pm (1.0E \text{ or } 1.3W) \tag{12-6}$$

where:

f_1 = 1.0 for floors in places of public assembly, for live loads in excess of 100 pounds per square foot (4.79 kN/m^2), and for garage live load.

= 0.5 for other live loads.

f_2 = 0.7 for roof configurations (such as saw tooth) that do not shed snow off the structure.

= 0.2 for other roof configurations.

Exceptions:

1. Factored load combinations for concrete per UBC §1909.2 and for masonry per UBC §2108.1.3 where load combinations do not include seismic forces.
2. Factored load combinations for this section multiplied by 1.1 for concrete and masonry where load combinations include seismic forces.
3. Where other factored load combinations are specifically required by the provisions of this code.

UBC §1612.3 Load Combinations Using Allowable Stress Design

UBC §1612.3.1 Basic Load Combinations. Where allowable stress design (working stress design) is used, structures and all portions thereof shall resist the most critical effects resulting from the following combinations of loads:

$$D \quad (12\text{-}7)$$

$$D + L + (L_r \text{ or } S) \quad (12\text{-}8)$$

$$D + (W \text{ or } E/1.4) \quad (12\text{-}9)$$

$$0.90D \pm E/1.4 \quad (12\text{-}10)$$

$$D + 0.75\,[L + (L_r \text{ or } S) + (W \text{ or } E/1.4)] \quad (12\text{-}11)$$

No increase in allowable stresses shall be used with these load combinations except as specifically permitted by §1612.3.2 and §1809.2.

UBC §1612.3.2 Alternative Basic Load Combinations. In lieu of the basic load combinations specified in §1612.3.1, structures and portions thereof shall be permitted to be designed for the most critical effects resulting from the following load combinations. When using these alternate basic load combinations, a one-third increase shall be permitted in allowable stresses for all combinations including W or E.

$$D + L + (L_r \text{ or } S) \quad (12\text{-}12)$$

$$D + L + (W \text{ or } E/1.4) \quad (12\text{-}13)$$

$$D + L + W + S/2 \quad (12\text{-}14)$$

$$D + L + S + W/2 \quad (12\text{-}15)$$

$$D + L + S + E/1.4 \quad (12\text{-}16)$$

$$0.9D \pm \mathrm{E}/1.4 \quad (12\text{-}16a)$$

Exceptions:

1. Crane hook loads need not be combined with roof live loads or with more than three fourths of the snow load or one half of the wind load.
2. Design snow loads of 30 psf (1.44 kN/m^2) or less need not be combined with seismic loads. Where design snow loads exceed 30 psf (1.44 kN/m^2) the design snow load shall be included with seismic loads, but may be reduced up to 75 percent where consideration of siting, configuration and load duration warrant when approved by the building official.

UBC §1612.4 Special Seismic Load Combinations. For both Allowable Stress Design and Strength Design, the following special load combinations for seismic design shall be used as specifically required by Chapter 16, Division IV, or by Chapters 18 through 23:

$$1.2D + f_1 L + 1.0E_m \quad (12\text{-}17)$$

$$0.9D \pm 1.0\,E_m \qquad (12\text{-}18)$$

where:

f_l = 1.0 for floors in places of public assembly, for live loads in excess of 100 pounds per square foot, (4.79 kN/m^2) and for garage live load.

= 0.5 for other live loads.

Division ~~III~~ IV Earthquake Design, UBC §1624 and §1625

Modify: UBC §1641 **General**

UBC §1641.1 Purpose. The purpose of the earthquake provisions herein is primarily to safeguard against major structural failures and loss of life, not to limit damage or maintain function.

UBC §1641.2 Minimum Seismic Design. Structures and portions thereof shall, as a minimum, be designed and constructed to resist the effects of seismic ground motions as provided in this ~~section~~ division.

UBC §1641.3 Seismic and Wind Design. When the code-prescribed wind design produces greater effects, the wind design shall govern, but detailing requirements and limitations prescribed in this and referenced sections shall be followed.

Modify: UBC §1642 **Definitions**

For the purposes of this section certain terms are defined as follows:

Base is the level at which the earthquake motions are considered to be imparted to the structure or the level at which the structure as a dynamic vibrator is supported.

Base Shear, V**,** is the total design lateral force or shear at the base of a structure.

Bearing Wall System is a structural system without a complete vertical load-carrying space frame. See §1644.6.2.

Boundary Element is an element at edges of openings or at perimeters of shear walls or diaphragms.

Braced Frame is an essentially vertical truss system of the concentric or eccentric type that is provided to resist lateral forces.

Building Frame System is an essentially complete space frame that provides support for gravity loads. See §1644.6.3.

Cantilevered Column Element in a lateral force resisting system column element that cantilevers from a fixed base and has minimal moment capacity at the top with lateral forces applied essentially at the top.

Collector is a member or element provided to transfer lateral forces from a portion of a structure to vertical elements of the lateral force resisting system.

Component is a part or element of an architectural, electrical, mechanical, or structural system.

Component, Equipment is a mechanical or electrical component or element that is part of a mechanical and/or electrical system.

Component, Flexible is a component, including its attachments, having a fundamental period greater than 0.06 second.

Component, Rigid is a component, including its attachments, having a fundamental period less than or equal to 0.06 second.

Concentrically Braced Frame is a braced frame in which the members are subjected primarily to axial forces.

Design Basis Ground Motion is that ground motion that has a 10 percent chance of being exceeded in 50 years as determined by a site specific hazard analysis or may be determined from a hazard map. A suite of ground motion time histories with dynamic properties representative of the site characteristics shall be used to represent this ground motion. The dynamic effects of the Design Basis Ground Motion may be represented by the Design Response Spectrum. See §1646.2.

Design Response Spectrum is an elastic response spectrum for 5% equivalent viscous damping used to represent the dynamic effects of the Design Basis Ground Motion for the design of structures in accordance with Sections 1645 and 1646. This response spectrum may be either a site specific spectrum based on geologic, tectonic, seismological and soil characteristics associated with a specific site or may be a spectrum constructed in accordance with the spectral shape in Figure 16-4 using the site specific values of C_a and C_v and multiplied by the acceleration of gravity, 386.4 in/sec^2 (9.815 m/sec^2). See §1646.2.

Diaphragm is a horizontal or nearly horizontal system acting to transmit lateral forces to the vertical resisting elements. The term "diaphragm" includes horizontal bracing systems.

Diaphragm or Shear Wall Chord is the boundary element of a diaphragm or shear wall that is assumed to take axial stresses analogous to the flanges of a beam.

Diaphragm Strut (drag strut, tie, collector) is the element of a diaphragm parallel to the applied load that collects and transfers diaphragm shear to the vertical resisting elements or distributes loads within the diaphragm. Such members may take axial tension or compression.

Drift. See "story drift."

Dual System is a combination of moment resisting frames and shear walls or braced frames designed in accordance with the criteria of §1644.6.5.

Eccentrically Braced Frame (EBF) is a steel-braced frame designed in conformance with §2211.10.

Elastic Response Parameters Forces and deformations determined from an elastic dynamic analysis using an unreduced ground motion representation, in accordance with §1646.

Essential Facilities are those structures that are necessary for emergency operations subsequent to a natural disaster.

Flexible Element or system is one whose deformation under lateral load is significantly larger than adjoining parts of the system. Limiting ratios for defining specific flexible elements are set forth in §1645.6. ~~1630.2~~

Horizontal Bracing System is a horizontal truss system that serves the same function as a diaphragm.

Intermediate Moment Resisting Frame (IMRF) is a concrete frame designed in accordance with §1921.8.

Lateral Force Resisting System is that part of the structural system designed to resist the Design Seismic Forces ~~lateral forces~~.

Moment Resisting Frame is a frame in which members and joints are capable of resisting forces primarily by flexure.

Moment Resisting Wall Frame (MRWF) is a masonry wall frame especially detailed to provide ductile behavior and designed in conformance with §2108.2.5.

Ordinary Braced Frame (OBF) is a steel-braced frame designed in accordance with the provisions of §2211.8 or 2212.6, or concrete braced frame designed in accordance with §1921.

Ordinary Moment Resisting Frame (OMRF) is a moment-resisting frame not meeting special detailing requirements for ductile behavior.

Orthogonal Effects are the earthquake load effects on structural elements common to the lateral force resisting systems along two orthogonal axes. ~~due to earthquake forces acting in a direction other than those axes.~~

Overstrength is a characteristic of structures where the actual strength is larger than the design strength. The degree of overstrength is material and system dependent.

P-Δ Effect is the secondary effect on shears, axial forces, and moments of frame members induced by the vertical loads acting on the laterally displaced building system ~~frame~~.

Shear Wall is a wall designed to resist lateral forces parallel to the plane of the wall (sometimes referred to as vertical diaphragm or structural wall).

Soft Story is one in which the lateral stiffness is less than 70 percent of the stiffness of the story above. See Table 16-L.

Space Frame is a three-dimensional structural system, without bearing walls, composed of members interconnected so as to function as a complete self-contained unit with or without the aid of horizontal diaphragms or floor bracing systems.

Special Concentrically Braced Frame (SCBF) is a steel-braced frame designed in conformance with the provisions of §2211.9.

Special Moment Resisting Frame (SMRF) is a moment-resisting frame specially detailed to provide ductile behavior and comply with the requirements given in Chapter 19 or 22.

Special Truss Moment Frame (STMF) is a moment-resisting frame specially detailed to provide ductile behavior and comply with the provisions of §2211.12.

Story is the space between levels. Story x is the story below level x.

Story Drift is the lateral displacement of one level relative to the level above or below.

Story Drift Ratio is the story drift divided by the story height.

Story Shear, V_x, is the summation of design lateral forces above the story under consideration.

Strength is the capacity of an element or a member to resist factored load as specified in Chapters 16, 18, 19, 21, and 22.

Structure is an assemblage of framing members designed to support gravity loads and resist lateral forces. Structures may be categorized as building structures or nonbuilding structures.

Subdiaphragm is a portion of a larger wood diaphragm designed to anchor and transfer local forces to primary diaphragm struts and the main diaphragm.

Vertical Load-Carrying Frame is a space frame designed to carry ~~all~~ vertical gravity loads.

Wall Anchorage System is the system of elements anchoring the wall to the diaphragm and those elements within the diaphragm required to develop the anchorage forces, including subdiaphragms and continuous ties, as specified in Sections 1648.2.8 and 1648.2.9.

Weak Story is one in which the story strength is less than 80 percent of that of the story above. See Table 16-L.

UBC §1626

Modify: **UBC §1643 Symbols and Notations** *Revise as follows:*

The following symbols and notations apply to the provisions of this section:

A_B = ground floor area of structure in square feet (m^2) to include area covered by all overhangs and projections.

A_c = the combined effective area, in square feet (m^2), of the shear walls in the first story of the structure.

A_e = the minimum cross sectional area in any horizontal plane in the first story, in square feet (m^2) of a shear wall.

a_p = numerical coefficient specified in §1647 and set forth in Table 16-O.

A_x = the torsional amplification factor at Level x.

~~C = numerical coefficient specified in §1628.2.1.~~

C_a = seismic coefficient, as set forth in Table 16-Q.

~~C_p = numerical coefficient specified in § 1630 and given in Table 16-O.~~

C_t = numerical coefficient given in §1645.2.2.

C_v = seismic coefficient, as set forth in Table 16-R.

D ~~L~~ = dead load on a structural element.

D_e = the length, in feet (m), of a shear wall in the first story in the direction parallel to the applied forces.

~~E = load due to an earthquake on a structural element.~~

E, E_h, E_m, E_v = earthquake loads set forth in §1645.1.

F_i, F_n, F_x = Design Seismic ~~lateral~~ Force applied to level i, n or x, respectively.

F_p = Design Seismic ~~lateral~~ Forces on a part of the structure.

F_{px} = Design Seismic Force on a diaphragm

F_t = that portion of the base shear, V, considered concentrated at the top of the structure in addition to F_n.

f_i = lateral force at Level i for use in Formula (45-11).

g = acceleration due to gravity.

h_i, h_n, h_x = height in feet (m) above the base to Level i, n or x, respectively.

I = importance factor given in Table 16-K.

I_p = importance factor specified in ~~§1630.2.~~ Table 16-K.

L ~~L~~ = live load on a structural element.

Level i = level of the structure referred to by the subscript i. "$i = 1$" designates the first level above the base.

Level n = that level uppermost in the main portion of the structure.

Level x = that level under design consideration. "x = 1" designates the first level above the base.

M = maximum moment magnitude.

N_a = near-source factor used in the determination of C_a in Seismic Zone 4 related to both the proximity of the building or structure to known faults with magnitudes and slip rates as set forth in Tables 16-S and 16-U.

N_v = near-source factor used in the determination of C_v in Seismic Zone 4 related to both the proximity of the building or structure to known faults with magnitudes and slip rates as set forth in Tables 16-T and 16-U.

PI = plasticity index of soil determined in accordance with approved national standards.

~~R_w = numerical coefficient given in Tables 16-N and 16-P.~~

r = a ratio used in determining ρ , see §1645.1.

R = $R_d R_o / I$

R_d = numerical coefficient representative of the global ductility capacity of lateral force resisting systems, as set forth in Table 16-N or 16-P. See Figure 16-3.

R_o = numerical coefficient, representative of the overstrength inherent in the lateral force resisting system, and given in Tables 16-N or 16-P. See Figure 16-3.

~~S = site coefficient for soil characteristics given in Table 16-J.~~

S_A, S_B, S_C, S_D, S_E, S_F = soil profile types as set forth in Table 16-J.

T = elastic fundamental period of vibration, in seconds, of the structure in the direction under consideration.

V = the total design lateral force or shear at the base given by Formula (45-6), (45-7), (45-8), or (45-12).

V_E = Elastic Response Base Shear illustrated in Figure 16-3.

V_M = lower bound system strength that is an estimate of the probable maximum base shear capacity of the lateral load resisting system illustrated in Figure 16-3.

V_x = the design story shear in Story x.

W = the total seismic dead load defined in §1645.1.1.

w_i, w_x = that portion of W located at or is assigned to level i or x, respectively.

W_p = the weight of an element or component.

w_{px} = the weight of the diaphragm and the element tributary thereto at Level x, including applicable portions of other loads defined in §1645.1.1.

Z = seismic zone factor as given in Table 16-I.

δ_i = horizontal displacement at Level i relative to the base due to applied lateral forces, f, for use in Formula (45-11).

Δ_E = Elastic Response Displacement corresponding to V_E and illustrated in Figure 16-3.

Δ_S = Design Level Response Displacement, which is the total drift or total story drift that occurs when the structure is subjected to the design seismic forces.

Δ_M = Maximum Inelastic Response Displacement, which is the total drift or total story drift that occurs when the structure is subjected to the Design Basis Ground Motion, including estimated elastic and inelastic contributions to the total deformation defined in §1645.9. See Figure 16-3.

ρ = Redundancy/Reliability Factor given by Formula (45-3).

Ω_0 = Seismic Force Amplification Factor, which is required to account for structural overstrength and set forth in §1645.3.1.

Modify: UBC §1627.1 *Revise as follows:*

UBC §1644 Criteria Selection

UBC §1644.1 Basis for Design. The procedures and the limitations for the design of structures shall be determined considering seismic zoning, site characteristics, occupancy, configuration, structural system and height in accordance with this section. Structures shall be designed with adequate strength to withstand the lateral displacements induced by the design basis ground motion, considering the inelastic response of the structure, and the inherent redundancy, overstrength and ductility of the lateral force resisting system. The minimum design strength shall be based on the Design Seismic Forces ~~shall be those~~ determined in accordance with the static lateral force procedure of §1645 except as modified by §1646.5.4. Where strength design is used, the load combinations of §1612.2 shall apply. Where Allowable Stress Design is used, the load combinations of §1612.3 shall apply. Allowable Stress Design may be used to evaluate sliding or overturning at the soil-structure interface regardless of the design approach used in the design of the structure provided load combinations of §1612.3 are utilized. One and two family dwellings in Seismic Zone 1 need not conform to the provisions of this section.

Modify: UBC §1627.2 *Delete and replace as follows:*

UBC §1644.2 Occupancy Categories. For purposes of earthquake-resistant design, each structure shall be placed in one of the occupancy categories listed in Table 16-K. Table 16-K ~~lists~~ assigns importance factors, I and I_p, and ~~review~~ structural observation requirements for each category.

Delete: UBC §1627.3 *Revise as follows:*

UBC §1644.3 Site Geology and Soil Characteristics. Each site shall be assigned a soil profile type based on properly substantiated geotechnical data using the site categorization procedure set forth in Division VI, §1662 and Table 16-J. ~~Site profile type and site coefficient, S, shall be established in accordance with Table 16-J.~~

> ***Exception***: When the soil properties are not known in sufficient detail to determine the Soil Profile Type, Type S_D shall be used. Soil Profile Types S_E or S_F need not be assumed unless the building official determines that Types S_E or S_F may be present at the site or in the event that Types S_E or S_F are established by geotechnical data.

UBC §1644.3.1 Soil Profile Type. Soil Profile Types S_A, S_B, S_C, S_D, S_E are defined in Table 16-J and Soil Profile Type S_F is defined as soils requiring site-specific evaluation as follows:

1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils.
2. Peats and/or highly organic clays where the thickness of peat or highly organic clay exceeds 10 feet (3048 mm).
3. Very high plasticity clays with a plasticity index, PI > 75, where the depth of clay exceeds 25 feet (7620 mm).
4. Very thick soft/medium stiff clays, where the depth of clay exceeds 120 feet (36576 mm).

Delete: UBC §1627.4 *Replace as follows:*

UBC §1644.4 Site Seismic Hazard Characteristics. Seismic hazard characteristics for the site shall be established based on the seismic zone and proximity of the site to active seismic sources, site soil profile characteristics and the structure's importance factor.

UBC §1644.4.1 Seismic Zone. Each site shall be assigned a seismic zone in accordance with Figure UBC §16-2. Each structure shall be assigned a seismic zone factor Z, in accordance with Table 16-I.

UBC §1644.4.2 Seismic Zone 4 Near-Source Factor. In Seismic Zone 4, each site shall be assigned a near-source factors in accordance with Tables 16-S and 16-T and the Seismic Source Type set forth in Table 16-U. The value of N_a used to determine C_a need not exceed 1.1 for structures complying with all of the following conditions:

1. The Soil Profile Type is S_A, S_B, S_C or S_D.
2. $\rho = 1.0$
3. Except in single story structures, Group R, Division 3 and Group U, Division 1 Occupancies, moment frame systems designated as part of the lateral force resisting system shall be special moment resisting frames.
4. The exceptions to §2211.7.5 shall not apply except for columns in one story buildings or columns at the top story of multi-story buildings.
5. None of the following structural irregularities are present: Type 1, 4 or 5 of Table 16-L and Type 1 or 4 of Table 16-M.

UBC §1644.4.3 Seismic Response Coefficients. Each structure shall be assigned a seismic coefficient, C_a, in accordance with Table 16-Q and a seismic coefficient, C_v, in accordance with Table 16-R.

Delete: UBC §1627.5 *Revise as follows:*

UBC §1644.5 Configuration Requirements

UBC §1644.5.1 General. Each structure shall be designated as being structurally regular or irregular in accordance with Sections 1644.5.2 and 1644.5.3.

UBC §1644.5.2 Regular Structures. Regular structures have no significant physical discontinuities in plan or vertical configuration or in their lateral force resisting systems such as the irregular features described ~~below~~ In §1644.5.3.

UBC §1644.5.3 Irregular Structures

1. Irregular structures have significant physical discontinuities in configuration or in their lateral force resisting systems. Irregular features include, but are not limited to, those described in Tables 16-L and 16-M. All ~~S~~structures in Seismic Zone 1 and ~~in~~ Occupancy Categor~~y~~ies 4 and 5 in Seismic Zone 2 need to be evaluated only for vertical irregularities of Type 5 (Table 16-L) and horizontal irregularities of Type 1 (Table 16-M).
2. Structures having ~~one or more~~ any of the features listed in Table 16-L shall be designated as having a vertical irregularity.

   Exception: Where no story drift ratio under design lateral forces is greater than 1.3 times the story drift ratio of the story above the structure may be deemed to not have the structural irregularities of Type 1 or 2 in Table 16-L. The story drift ratio for the top two stories need not be considered. The story drifts for this determination may be calculated neglecting torsional effects.
3. Structures having ~~one or more~~ any of the features listed in Table 16-M shall be designated as having a plan irregularity.

Modify: UBC §1627.6 *Revise as follows:*

UBC §1644.6 Structural Systems

UBC §1644.6.1 General. Structural systems shall be classified as one of the types listed in Table 16-N and defined in this subsection.

UBC §1644.6.2 Bearing Wall System. A structural system without a complete vertical load-carrying space frame. Bearing walls or bracing systems provide support for all or most gravity loads. Resistance to lateral loads is provided by shear walls or braced frames.

UBC §1644.6.3 Building Frame System. A structural system with an essentially complete space frame providing support for gravity loads. Resistance to lateral load is provided by shear walls or braced frames.

UBC §1644.6.4 Moment-Resisting Frame System. A structural system with an essentially complete space frame providing support for gravity loads. Moment-resisting frames provide resistance to lateral load primarily by flexural action of members.

UBC §1644.6.5 Dual System. A structural system with the following features:

1. An essentially complete space frame that provides support for gravity loads.
2. Resistance to lateral load is provided by shear walls or braced frames and moment-resisting frames (SMRF, IMRF, MMRWF or steel OMRF). The moment resisting frames shall be designed to independently resist at least 25 percent of the design base shear.

3. The two systems shall be designed to resist the total design base shear in proportion to their relative rigidities considering the interaction of the dual system at all levels.

UBC §1644.6.6 Cantilevered Column System. A structural system relying on cantilevered column elements for lateral resistance.

UBC §1644.6.7 Undefined Structural System. A structural system not listed in Table 16-N.

UBC §1644.6.8 Nonbuilding Structural System. A structural system conforming to §1649.

Modify: **UBC §1627.7 and §1627.8** *Revise as follows:*

UBC §1644.7 Height Limits. Height limits for the various structural systems in Seismic Zones 3 and 4 are given in Table 16-N.

> *Exception:* Regular structures may exceed these limits by not more than 50 percent for unoccupied structures that are not accessible to the general public.

UBC §1644.8 Selection of Lateral Force Procedure

UBC §1644.8.1 General. Any structure may be, and certain structures defined below shall be, designed using the dynamic lateral force procedures of §1646.

UBC §1644.8.2 Simplified Static. Except in structures using cantilevered column systems, the simplified static lateral force procedure set forth in § 1645.2.3 may be used for the following structures of Occupancy Categories 4 or 5:

1. Single-family dwellings not over two stories in height excluding basements.
2. Buildings of any occupancy (including single-family dwellings) not more than three stories in height excluding basements that use light frame construction.
3. Other regular structures not more than two stories in height excluding basements.

UBC §1644.8.~~2~~3 Static. The static lateral force procedure of §1645 may be used for the following structures:

1. All structures, regular or irregular, in Seismic Zone 1 and in Occupancy Categories 4 and 5 in Seismic Zone 2.
2. Regular structures under 240 feet (73 152 mm) in height with lateral force resistance provided by systems listed in Table 16-N except where §1644.8.4, Item 4, applies.
3. Irregular structures not more than five stories or 65 feet (19 812 mm) in height.
4. Structures having a flexible upper portion supported on a rigid lower portion where both portions of the structure considered separately can be classified as being regular, the average story stiffness of the lower portion is at least 10 times the average story stiffness of the upper portion and the period of the entire structure is not greater than 1.1 times the period of the upper portion considered as a separate structure fixed at the base.

UBC §1644.8.~~3~~4 Dynamic. The dynamic lateral force procedure of §1646 shall be used for all other structures, including the following:

1. Structures 240 feet (73 152 mm) or more in height except as permitted by §1644.8.3, Item 1.
2. Structures having a stiffness, weight, or geometric vertical irregularity of Type 1, 2, or 3 as defined in Table 16-L or structures having irregular features not described in Tables 16-L or 16-M except as permitted by §1645.4.2.
3. Structures over five stories or 65 feet (19 812 mm) in height in Seismic Zone 3 and 4 not having the same structural system throughout their height except as permitted by §1645.4.2.
4. Structures, regular or irregular, located on Soil Profile Type ~~S4~~ S_F that have a period greater than 0.7 second. The analysis shall include the effects of soils at the site and shall conform to §1646.2, Item 4.

Modify: **UBC §1627.9 and UBC §1627.10** *Revise as follows:*

UBC §1644.9 System Limitations

UBC §1644.9.1 Discontinuity. Structures with a discontinuity in capacity, vertical irregularity Type 5 as defined in Table 16-L, shall not be over two stories or 30 feet (9144 mm) in height where the weak story has a calculated strength of less than 65 percent of the story above.

> ***Exception:*** Where the weak story is capable of resisting a total lateral seismic force of ~~$3(R_w/8)$~~ Ω_o times the design seismic force prescribed in §1645.

UBC §1644.9.2 Undefined Structural Systems. ~~Undefined structural systems shall be shown by technical and test data which establish the dynamic characteristics and demonstrate the lateral force resistance and energy absorption capacity to be equivalent to systems listed in Table 16-N for equivalent R_w values.~~ For undefined structural systems not listed in Table 16-N, the coefficients R_o and R_d shall be substantiated by approved cyclic test data and analyses. The following items shall be addressed when establishing R_o and R_d:

1. Dynamic response characteristics.
2. Lateral force resistance.
3. Overstrength and strain hardening or softening.
4. Strength and stiffness degradation.
5. Energy dissipation characteristics.
6. System ductility.
7. Redundancy.

UBC §1644.9.3 Irregular Features. All structures having irregular features described in Table 16-L, or 16-M shall be designed to meet the additional requirements of those section referred to in the tables.

UBC §1644.10 Alternative Procedures

UBC §1644.10.1 General. Alternative lateral force procedures using rational analyses based on well established principles of mechanics may be used in lieu of those prescribed in these provisions.

UBC §1644.10.2 Seismic Isolation. Seismic isolation, energy dissipation and damping systems may be used in the design of structures when approved by the building official and when special detailing ~~are~~ is used to provide results equivalent to those obtained by the use of conventional structural systems. For alternate design procedures on seismic isolation systems, refer to Appendix Chapter 16, Division III, Earthquake Regulations for Seismic-Isolated Structures.

UBC §1645 Minimum Design Lateral Forces and Related Effects

Modify: UBC §1628.1 *Revise as follows:*

UBC §1645.1—Earthquake Loads and Modeling Requirements

UBC §1645.1.1 ~~General~~Earthquake Loads. Structures shall be designed for ground motion producing structural response and seismic forces ~~coming from~~ in any horizontal direction. The following earthquake loads shall be used in the load combinations set forth in §1612:

$$E = \rho E_h + E_v \qquad (45\text{-}1)$$

$$E_m = \Omega_o E_h \qquad (45\text{-}2)$$

where:

E = the earthquake load on an element of the structure resulting from the combination of the horizontal component, E_h, and the vertical component, E_v.

E_h = the earthquake load due to the base shear, V, as set forth in §1645.2 or the design lateral force, F_p, as set forth in §1647.

E_m = the estimated maximum earthquake force that can be developed in the structure as set forth in §1645.1.1.

E_v = the load effect resulting from the vertical component of the earthquake ground motion and is equal to an addition of $0.5C_aID$ to the dead load effect, D, for Strength Design, and may be taken as zero for Allowable Stress Design.

Ω_o = is the seismic force amplification factor required to account for structural overstrength and is set forth in §1645.3.1.

ρ = Reliability/Redundancy Factor as given by the following formula:

$$\rho = 2 - \frac{20}{r_{max}\sqrt{A_B}} \qquad (45\text{-}3)$$

for SI:

$$= 2 - \frac{6.1}{r_{max}\sqrt{A}}$$

where:

r_{max} = maximum element-story shear ratio.

For a given direction of loading, the element-story shear ratio is the ratio of the design story shear in the most heavily loaded single element divided by the total design story shear. For any given story level i, the element-story shear ratio is denoted as r_i. The maximum element-story shear ratio r_{max} is defined as the largest of the element story shear ratios, r_i, which occurs in any of the story levels at or below the two-thirds height level of the building.

For braced frames the value of r_i is equal to the maximum horizontal force component in a single brace element divided by the total story shear.

For moment frames r_i shall be taken as the maximum of the sum of the shears in any two adjacent columns in a moment frame bay divided by the story shear. For columns common to two bays with moment resisting connections on opposite sides at level i in the direction under consideration, 70% of the shear in that column may be used in the column shear summation.

For shear walls, r_i shall be taken as the maximum value of the product of the wall shear multiplied by $10/l_w$ (Sl.$3.05/l_w$) and divided by the total story shear, where l_w is the length of the wall in feet (m).

For dual systems, r_i shall be taken as the maximum value of r_i as defined above considering all lateral load resisting elements. The lateral loads shall be distributed to elements based on relative rigidities considering the interaction of the dual system. For dual systems, the value of ρ need not exceed 80 percent of the value calculated above.

ρ shall not be taken less than 1.0 and need not be greater than 1.5 and A_B is the ground floor area of the structure in square feet (m^2). For Special Moment Resisting Frames, except when used in dual systems, ρ shall not exceed 1.25. The number of bays of Special Moment Resisting Frames shall be increased to reduce r such that ρ is less than or equal to 1.25.

> ***Exception:*** A_B may be taken as the average floor area in the upper setback portion of the building where a larger base area exists at ground floor.

When calculating drift, or when the structure is located in Seismic Zones 0, 1 or 2, ρ shall be taken equal to one.

The ground motion producing lateral response and design seismic forces may be assumed to act nonconcurrently in the direction of each principal axis of the structure, except as required by §1648.1.

Seismic dead load, W, is the total dead load and applicable portions of other loads listed below.

1. In storage and warehouse occupancies, a minimum of 25 percent of the floor live load shall be applicable.
2. Where a partition load is used in the floor design, a load of not less than 10 pounds per square foot (psf) (0.48 kN/m^2) shall be included.
3. Design snow loads of 30 pounds per square foot (psf) (1.44 kN/m^2)or less need not be included. Where design snow loads exceed 30 psf (1.44 kN/m^2) the design snow load shall be included, but may be reduced up to 75 percent where considerations of siting, configuration and load duration warrant when approved by the building official.
4. Total weight of permanent equipment shall be included.

UBC §1645.1.2 Modeling Requirements. The mathematical model of the physical structure shall include all elements of the lateral force resisting system. The model shall also include the stiffness and strength of elements significant to the distribution of forces and shall represent the spatial distribution of the mass and stiffness of the structure. In addition, the model shall comply with the following:

1. Stiffness properties of reinforced concrete and masonry elements shall consider the effects of cracked sections.
2. For steel moment frame systems, the contribution of panel zone deformations to overall story drift shall be included.

Modify: UBC §1628.9 *Relocate to §1645.1.3 and revise as follows:*

~~UBC §1628.9~~UBC §1645.1.3 P-delta Effects. The resulting member forces and moments and the story drifts induced by P-Δ effects shall be considered in the evaluation of overall structural frame stability and shall be evaluated using the forces producing the displacements of Δ_S. P-Δ need not be considered when the ratio of secondary moment to primary moment does not exceed 0.10; the ratio may be evaluated for any story as the product of the total dead, floor live and snow load, as required in §1612, above the story times the seismic drift in that story divided by the product of the seismic shear in that story times the height of that story. In Seismic Zones 3 and 4, P-Δ need not be considered when the story drift ratio does not exceed ~~0.02/R_w~~ 0.02I/$R_d R_o$~~.~~

Delete: UBC §1628.2.1 *Replace as follows:*

UBC §1645.2 Static Force Procedure

UBC §1645.2.1 Design Base Shear. The total design base shear in a given direction shall be determined from the following formulas:

$$V = \frac{C_v}{RT} W \qquad (45\text{-}4)$$

where:

$$R = \frac{R_d R_o}{I} \tag{45-5}$$

The total design base shear need not exceed the following:

$$V = \frac{2.5C_a}{R}W \tag{45-6}$$

The total design base shear shall not be less than the following:

$$V = 0.11C_a IW \tag{45-7a}$$

Additionally, in Seismic Zone 4, the total base shear shall also not be less than the following:

$$V = \frac{0.8ZN_v}{R}W \tag{45-7b}$$

Modify: UBC §1628.2.2 *Revise as follows:*

UBC §1645.2.2 Structure Period. The value of T shall be determined from one of the following methods:

1. **Method A.** For all buildings, the value T may be approximated from the following formula:

$$T = C_t(h_n)^{3/4} \tag{45-8}$$

where:

C_t = 0.035 (0.0853) for steel moment-resisting frames.

C_t = 0.030 (0.0731) for reinforced concrete moment-resisting frames and eccentrically braced frames.

C_t = 0.020 (0.0488) for all other buildings.

Alternatively, the value of C_t for structures with concrete or masonry shear walls may be taken as $0.1/\sqrt{A_c}$ (For **SI**: $0.0743/\sqrt{A_c}$ for A_c in m^2).

The value of A_c shall be determined from the following formula:

$$A_c = \sum A_c[0.2 + (D_e/h_n)^2] \tag{45-9}$$

The value of D_e/h_n used in formula (45-9) shall not exceed 0.9.

2. **Method B.** The fundamental period T may be computed using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis. The analysis shall be in accordance with the requirements of §1645.1.2. ~~This requirement may be satisfied by using the following formula:~~ The value of T from Method B shall not exceed a value 30

percent greater than the value of T obtained from Method A in Seismic Zone 4 and 40 percent in Seismic Zones 1, 2 and 3.

The fundamental period T may be computed by using the following formula:

$$T = 2\pi\sqrt{\left(\sum_{i=1}^{n} w_i\delta_i^2\right) \div \left(g\sum_{i=1}^{n} f_i\delta_i\right)} \qquad (45\text{-}10)$$

The values of f_i represent any lateral force distributed approximately in accordance with the principles of Formulas (45-15), (45-16) and (45-17) or any other rational distribution. The elastic deflections δ_i shall be calculated using the applied lateral forces f_i. ~~The value of T from Method B shall not be over 30 percent greater than the value of T obtained from Method A in Seismic Zone 4 and 40 percent in Seismic Zones 1, 2 and 3.~~

Add new sections: **UBC §1645.2.3 and UBC §1645.3 Simplified Design Base Shear.** *Add new sections and renumber accordingly:*

UBC §1645.2.3.1 General. Structures conforming to the requirements of §1644.8.2 may be designed using this procedure.

UBC §1645.2.3.2 Base Shear. The total design base shear in a given direction shall be determined from the following formula:

$$V = 0.8C_aW \qquad (45\text{-}11)$$

where the value of C_a shall be based on Table 16-Q for the Soil Profile Type and a Near Source Factor $N_a = 1.0$. When the soil properties are not known in sufficient detail to determine the Soil Profile Type, Type S_D shall be used in Seismic Zones 3 and 4 and Type S_E shall be used in Seismic Zones 1, 2A and 2B.

UBC §1645.2.3.3 Vertical Distribution. The forces at each level shall be calculated using the following formula:

$$F_x = 0.8C_aW \qquad (45\text{-}12)$$

where the value of C_a shall be based on Table 16-Q for the Soil Profile Type and a Near Source Factor $N_a = 1.0$. When the soil properties are not known in sufficient detail to determine the Soil Profile Type, Type S_D may be used in Seismic Zones 3 and 4 and Type S_E shall be used in Seismic Zones 1, 2A and 2B.

UBC §1645.2.3.4 Applicability. Sections 1645.1.2, 1645.1.3, 1645.2.1, 1645.2.2, 1645.5, 1645.9, 1645.10 and 1646 shall not apply when using the simplified procedure. Where required, Ω_0 shall be equal to 3.0. Where used, Δ_M shall be taken equal to 0.01 times the story height of all stories. In §1648.2.9, Formula (48-1) shall read $F_{px} = 0.8C_aW_p$

UBC §1645.3 Determination of Seismic Factors

UBC §1645.3.1 Determination of Ω_o**.** For specific elements of the structure, as specifically identified in this code, the minimum design strength shall be the product of the seismic force overstrength factor Ω_0 and the design seismic forces set forth in §1645. For both Allowable Stress Design and Strength Design, the Seismic Force Overstrength Factor, Ω_0 shall be determined as follows:

$$\Omega_o = 1.1R_o \qquad (45\text{-}13)$$

UBC §1645.3.2 Determination of R_0 ***and*** R_d**.** The terms R_0 and R_d shall be taken from Table 16-N.

Modify: UBC §1628.3 *Revise as follows:*

UBC §1645.4 Combinations of Structural Systems

UBC §1645.4.1 General. Where combinations of structural systems are incorporated into the same structure, the requirements of this subsection shall be satisfied.

UBC §1645.4.2 Vertical Combinations. The value of ~~R_w~~ R used in the design of any story shall be less than or equal to the value of ~~R_w~~ R used in the given direction for the story above.

> ***Exception:*** This requirement need not be applied to a story where the dead weight above that story is less than 10 percent of the total dead weight of the structure.

Structures may be designed using the procedures of this section under the following conditions:

11. The entire structure is designed using the lowest ~~R_w~~ R of the lateral force-resisting systems used or,
2. The following two-stage static analysis procedures may be used for structures conforming to §1644.8.3, Item 4.
    - 2.1. The flexible upper portion shall be designed as a separate structure, supported laterally by the rigid lower portion, using the appropriate values of ~~R_w~~ R and ρ.
    - 2.2. The rigid lower portion shall be designed as a separate structure using the appropriate values of ~~R_w~~ R and ρ. The reactions from the upper portion shall be those determined from the analysis of the upper portion amplified by the ratio of the ~~R_w~~ product (R_dR_o/ρ) of the upper portion over the ~~R_w~~ product (R_dR_o/ρ) of the lower portion.

UBC §1645.4.3 Combinations Along Different Axes. In Seismic Zones 3 and 4 where a structure has a bearing wall system in only one direction, the value of ~~R_w~~ R

used for design in the orthogonal direction shall not be greater than that used for the bearing wall system.

Any combination of bearing wall systems, building frame systems, dual systems, or moment-resisting frame systems may be used to resist seismic forces in structures less than 160 feet (48 768 mm) in height. Only combinations of dual systems and special moment-resisting frames shall be used to resist seismic forces in structures exceeding 160 feet (48 768 mm) in height in Seismic Zones 3 and 4.

UBC §1645.4.4 Combinations Along the Same Axes. For other than dual systems, where a combination of different structural systems is utilized to resist lateral forces in the same direction, the value of ~~R_w~~ R used in that direction shall not be greater than the least values ~~of~~ for any of the systems utilized in that same direction.

Modify: UBC **§1628.4** *Revise as follows:*

UBC §1645.5 Vertical Distribution of Force. The total force shall be distributed over the height of the structure in conformance with Formulas (45-15), (45-16) and (45-17) in the absence of a more rigorous procedure.

$$V = F_t + \sum_{i=1}^{n} F_i \tag{45-15}$$

The concentrated force F_t at the top, which is in addition to F_n, shall be determined from the formula:

$$F_t = 0.07TV \tag{45-16}$$

The value of T used for the purpose of calculating F_t ~~may~~ shall be the period that corresponds with the design base shear as computed using Formula (45-4). F_t need not exceed 0.25V and may be considered as zero where T is 0.7 seconds or less. The remaining portion of the base shear shall be distributed over the height of the structure, including Level n, according to the formula:

$$F_x = \frac{(V - F_t)w_x h_x}{\sum_{i=1}^{n} w_i h_i} \tag{45-17}$$

At each level designated as x, the force F_x shall be applied over the area of the building in accordance with the mass distribution at that level. ~~Stresses in each structural element~~ Structural displacements and design seismic forces shall be calculated as the effect of forces F_x and F_t applied at the appropriate levels above the base.

Modify: **UBC §1628.5 and UBC §1628.6** *Change section numbers as follows:*

UBC §1645.~~5~~6 Horizontal Distribution of Shear. The design story shear V_x, in any story is the sum of the forces F_t and F_x above that story. V_x shall be distributed to the various elements of the vertical lateral force resisting system in proportion to their rigidities, considering the rigidity of the diaphragm. See §1648.2.4 for rigid elements that are not intended to be part of the lateral force resisting systems.

Where diaphragms are not flexible, the mass at each level shall be assumed to be displaced from the calculated center of mass in each direction a distance equal to 5 percent of the building dimension at that level perpendicular to the direction of the force under consideration. The effect of this displacement on the story shear distribution shall be considered.

Diaphragms shall be considered flexible for the purposes of distribution of story shear and torsional moment when the maximum lateral deformation of the diaphragms is more than two times the average story drift of the associated story. This may be determined by comparing the computed midpoint in-plane deflection of the diaphragm itself under lateral load with the story drift of adjoining vertical resisting elements under equivalent tributary lateral load.

UBC §1645.~~6~~7 Horizontal Torsional Moments. Provisions shall be made for the increased shears resulting from horizontal torsion where diaphragms are not flexible. The most severe load combination for each element shall be considered for design.

The torsional design moment at a given story shall be the moment resulting from eccentricities between applied design lateral forces at levels above that story and the vertical resisting elements in that story plus an accidental torsion.

The accidental torsional moment shall be determined by assuming the mass is displaced as required by §1645.6.

Where torsional irregularity exists, as defined in Table 16-M, the effects shall be accounted for by increasing the accidental torsion at each level by an amplification factor A_x, determined from the following formula:

$$A_x = \left[\frac{\delta_{max}}{1.2\delta_{avg}}\right]^2 \qquad (45\text{-}18)$$

where:

δ_{avg} = the average of the displacements at the extreme points of the structure at Level x.

δ_{max} = the maximum displacement at Level x.

The value of A_x need not exceed 3.0.

Modify: UBC §1628.7 *Revise as follows:*

UBC §1645.8 Overturning

UBC §1645.8.1 General. Every structure shall be designed to resist the overturning effects caused by earthquake forces specified in §1645.5. At any level, the overturning moments to be resisted shall be determined using those seismic forces (F_t and F_x) that act on levels above the level under consideration. At any level, the incremental changes of the design overturning moment shall be distributed to the various resisting elements in the manner prescribed in §1645.6. Overturning effects on every element shall be carried down to the foundation. See ~~§1631~~ §1612 and §1644.1 for combining gravity and seismic forces.

UBC §1645.8.2 Seismic Zones 3 and 4. In Seismic Zones 3 and 4, where a lateral load-resisting element is discontinuous, such as for vertical irregularity Type 4 in Table 16-L or plan irregularity Type 4 in Table 16-M, columns supporting such elements shall have the strength to resist the axial force resulting from the ~~following~~ load combinations of §1612.4, in addition to all other applicable load combinations.

~~1.0 DL + 0.8 LL + 3(Rw/8)E~~

~~0.85 DL +/- 3(Rw/8)E~~

1. The axial forces in such columns need not exceed the capacity of other elements of the structure to transfer such loads to the columns.
2. Such columns shall be capable of carrying the above-described axial forces without exceeding the axial load strength of the column. ~~For designs using working stress methods this capacity may be determined using an allowable stress increase of 1.7. The additional one-third allowable stress increase per §1603.5 is not applicable.~~
3. Such columns shall meet the following detailing or member limitations:

Chapter 19, §1921.4, for concrete and Chapter 22, §2211.5 for steel in structures in Seismic Zones 3 and 4.

Chapter 19, §1921.8, for concrete and Chapter 22, Divisions I and IX, special provisions for developing plastic hinges at ultimate loading, for steel in structures in Seismic Zone 2.

UBC §1645.8.3 At Foundation. See §1644.1 and §1809.4 for overturning moments to be resisted at the foundation soil interface.

Add new section: UBC §1628.8 *Add a section before Section UBC §1628.8 and renumber subsequent sections accordingly:*

UBC §1645.9 Drift. Drift or horizontal displacements of the structure shall be computed where required by this code. For both Allowable Strength Design and Strength Design, the Maximum Inelastic Response Displacement, Δ_M, of the structure caused by the Design Basis Ground Motion shall be determined in accordance with this section. The drifts corresponding to the design seismic forces of

§1645.2.1, Δ_S, shall be determined in accordance with §1645.9.1. To determine Δ_M, these drifts shall be amplified in accordance with §1645.9.2.

UBC §1645.9.1 Determination of Δ_S. A static, elastic analysis of the lateral force resisting system shall be prepared using the design seismic forces from §1645.2.1. Alternatively, dynamic analysis may be performed in accordance with §1646. Where Allowable Stress Design is used and where drift is being computed, the load combinations of §1612.2 shall be used. The mathematical model shall comply with §1645.1.2. The resulting deformations, denoted as Δ_S, shall be determined at all critical locations in the structure. Calculated drift shall include translational and torsional deflections.

UBC §1645.9.2 Determination of Δ_M. The Maximum Inelastic Response Displacements, Δ_M, shall be computed as follows:

$$\Delta_M = 0.7 R_d R_o \Delta_s \qquad (45\text{-}19)$$

> *Exception:* Alternatively, Δ_M may be computed by nonlinear time history analysis in accordance with §1646.6.

The analysis used to determine the Maximum Inelastic Response Displacement Δ_M shall consider P-Δ effects.

Modify: UBC **§1628.8** *Revise as follows:*

UBC §1645.10 Story Drift Limitation

UBC §1645.10.1 ~~**Defined**. Story drift is the displacement of one level relative to the level above or below due to the design lateral forces. Calculated drift shall include deflections due to translation and torsional effects.~~ Story drifts shall be computed using the Maximum Inelastic Response Displacement, Δ_M.

UBC §1645.10.2 ~~**Calculated.**~~ Calculated story drift using Δ_M, shall not exceed ~~$0.04/R_w$ or 0.005~~ 0.025 times the story height for structures having a fundamental period of less than 0.7 second. For structures having a fundamental period of 0.7 seconds or greater, the calculated story drift shall not exceed ~~$0.03/R_w$ or 0.004~~ 0.020 times the story height.

Exceptions:

1. These drift limits may be exceeded when it is demonstrated that greater drift can be tolerated by both structural elements and nonstructural elements that could affect life safety. The drift used in this assessment shall be based upon the Maximum Inelastic Response Displacement Δ_M.
2. There shall be no drift limit in single-story steel framed structures classified as Groups B, F, M and S Occupancies or Group H, Division 4 or 5 Occupancies. In Group B Occupancies, the primary use shall be limited to workshops. Minor accessory uses shall be

allowed in accordance with the provisions of §302. Structures on which this exception is used shall not have equipment attached to the structural frame or shall have such equipment detailed to accommodate the additional drift. Walls that are laterally supported by the steel frame shall be designed to accommodate the drift, in accordance with §1648.2.4.

UBC §1645.10.3 ~~***Deriving Forces.***~~ The design lateral forces used to determine the calculated drift may disregard the limitations of Eq. 45-7a and Eq. 45-7b and may be ~~derived from a value of C~~ based on the period determined from Formula ~~(28-5)~~ (45-10) neglecting ~~the lower-bound ratio for C/R_w of 0.075 of §1628.1 and~~ the limitation of §1645.2.2, Item 2.

Delete: **UBC §1628.9**

Delete: **UBC §1628.10** *Replace as follows:*

UBC §1645.11 Vertical Component. The following requirements apply in Seismic Zones 3 and 4 only. Horizontal cantilever components shall be designed for a net upward force of ~~$0.5ZW_p$.~~ $0.7C_aIW_p$.

In addition to all other applicable load combinations, horizontal prestressed components shall be designed using not more than 50 percent of the dead load for the gravity load, alone or in combination with the lateral force effects.

Dynamic Analysis Procedures

Modify: **UBC §1629 Dynamic ~~Lateral-force~~ Analysis Procedures** *Revise as follows:*

UBC §1646.1 General. Dynamic analyses procedures, when used, shall conform to the criteria established in this section. The analysis shall be based on an appropriate ground motion representation and shall be performed using accepted principles of dynamics. Structures that are designed in accordance with this section shall comply with all other applicable requirements of these provisions.

UBC §1646.2 Ground Motion. The ground motion representation shall, as a minimum, be one having a 10 percent probability of being exceeded in 50 years, shall not be reduced by the quantity R and may be one of the following:

1. ~~The normalized response spectrum given in Figure 16-4.~~ An elastic design response spectrum constructed in accordance with Figure 16-4, using the values of C_a and C_v consistent with the specific site. The design acceleration ordinates shall be multiplied by the acceleration of gravity, 386.4 in/sec^2 (9.815 m/sec^2).
2. A site-specific elastic design response spectrum based on geologic, tectonic, seismologic and soil characteristics associated with the specific site. The spectrum shall be developed for a damping ratio of 0.05 unless a different value is shown to be consistent with the anticipated structural behavior at the intensity of shaking established for the site.

3. Ground motion time histories developed for the specific site shall be representative of actual earthquake motions. Response spectra from time histories, either individually or in combination, shall approximate the site design spectrum conforming to §1646.2, Item 2.
4. For structures on Soil Profile Type ~~S4~~ S_F, the following requirements shall apply when required by §1644.8.4, Item 4.
    - 4.1 The ground motion representation shall be developed in accordance with Items 2 and 3 above.
    - 4.2 Possible amplification of building response due to the effects of soil-structure interaction and lengthening of building period caused by inelastic behavior shall be considered.
    - ~~4.3 The base shear determined by these procedures may be reduced to a design base shear, V, by dividing by a factor not greater than the appropriate R_w factor for the structure.~~
5. The vertical component of ground motion may be defined by scaling corresponding horizontal accelerations by a factor of two-thirds. Alternative factors may be utilized when substantiated by site-specific data. Where the Near Source Factor, N_a is greater than 1.0, site specific vertical response spectra shall be used in lieu of the factor of two-thirds.

UBC §1646.3 Mathematical Model. A mathematical model of the physical structure shall represent the spatial distribution of the mass and stiffness of the structure to an extent adequate for the calculation of the significant features of its dynamic response. A three-dimensional model shall be used for the dynamic analysis of structures with highly irregular plan configurations such as those having a plan irregularity defined in Table 16-M and having a rigid or semi-rigid diaphragm. The stiffness properties used in the analysis and general mathematical modeling shall be in accordance with §1645.1.2.

UBC §1646.4 Description of Analysis Procedures

UBC §1646.4.1 Response Spectrum Analysis. An elastic dynamic analysis of a structure utilizing the peak dynamic response of all modes having a significant contribution to total structural response. Peak modal responses are calculated using the ordinates of the appropriate response spectrum curve that correspond to the modal periods. Maximum modal contributions are combined in a statistical manner to obtain an approximate total structural response.

UBC §1646.4.2 Time-History Analysis. An analysis of the dynamic response of a structure at each increment of time when the base is subjected to a specific ground motion time history.

UBC §1646.5 Response Spectrum Analysis

UBC §1646.5.1 Response Spectrum Representation and Interpretation of Results. The ground motion representation shall be in accordance with §1646.2. The corresponding response parameters, including forces, moments and displacements, shall be denoted as Elastic Response Parameters. Elastic Response Parameters may be reduced in accordance with §1646.5.4.

UBC §1646.5.2 Number of Modes. The requirement of §1646.4.1 that all significant modes be included may be satisfied by demonstrating that for the modes considered, at least 90 percent of the participating mass of the structure is included in the calculation of response for each principal horizontal direction.

UBC §1646.5.3 Combining Modes. The peak member forces, displacements, story forces, story shears, and base reactions for each mode shall be combined by recognized methods. When three dimensional models are used for analysis, modal interaction effects shall be considered when combining modal maxima.

UBC §1646.5.4 ~~Scaling of Results~~ Reduction of Elastic Response Parameters for Design. ~~The base shear for a given direction determined using these procedures, when less than the values below, shall be scaled up to these values.~~ Elastic Response Parameters may be reduced for purposes of design in accordance with the following. Items 1, 2 and 3: with the limitation that in no case shall the Elastic Response Parameters be reduced such that the corresponding design base shear is less than the Elastic Response Base Shear divided by the value of R.

1. For all regular structures where the ground motion representation complies with §1646.2, Item 1, Elastic Response Parameters may be reduced such that the corresponding design base shear is not less than 90 percent of the base shear determined in accordance with §1645.2.
2. For all regular structures where the ground motion representation complies with §1646.2, Item 2, Elastic Response Parameters may be reduced such that the corresponding design base shear is not less than 80 percent of the base shear determined in accordance with §1645.2.
3. For all irregular structures, regardless of the ground motion representation, Elastic Response Parameters may be reduced such that the corresponding design base shear is not less than 100 percent of the base shear determined in accordance with §1645.2.

The corresponding reduced design seismic forces shall be used for design in accordance with §1612.

~~1.~~ ~~The base shear shall be increased to the following percentages of the values determined from the procedures of §1628:~~

~~1.1~~ ~~One hundred percent for irregular buildings.~~

~~1.2~~ ~~Ninety percent for regular buildings, except that the base shear shall not be less than 80 percent of that determined from §1628 using the period *T*, calculated from method A.~~ All corresponding response parameters, including deflection, member forces and moments, shall be increased proportionately.

UBC §1646.5.5 Directional Effects. Directional effects for horizontal ground motion shall conform to the requirements of §1645.1. The effects of vertical ground motions on horizontal cantilevers and prestressed elements shall be considered in accordance with §1645.11. Alternately, vertical seismic response may be determined by dynamic response methods; in no case shall the response used for design be less than that obtained by the static method.

UBC §1646.5.6 Torsion. The analysis shall account for torsional effects, including accidental torsional effects as prescribed in §1645.7. Where three-dimensional models are used for analysis, effects of accidental torsion shall be accounted for by appropriate adjustments in the model such as adjustment of mass locations, or by equivalent static procedures such as provided in §1645.6.

UBC §1646.5.7 Dual Systems. Where the lateral forces are resisted by a dual system as defined in §1644.6.5, the combined system shall be capable of resisting the base shear determined in accordance with this section. The moment-resisting frame shall conform to §1644.6.5, Item 2 and may be analyzed using either the procedures of §1645.5 or those of §1646.5.

UBC §1646.6 Time-History Analysis. ~~Time-history analyses shall meet the requirements of §1627.10.~~

UBC §1646.6.1 Time-History. Time-history analysis shall be performed with pairs of appropriate horizontal ground motion time-history components, which shall be selected and scaled from not less than three recorded events. Appropriate time histories shall have magnitudes, fault distances and source mechanisms that are consistent with those that control the design basis earthquake (or maximum capable earthquake). Where three appropriate recorded ground motion time history pairs are not available, appropriate simulated ground motion time history pairs may be used to make up the total number required. For each pair of horizontal ground motion components, the square root of the sum of the squares (SRSS) of the 5 percent-damped site specific spectrum of the scaled horizontal components shall be constructed. The motions shall be scaled such that the average value of the SRSS spectra does not fall below 1.4 times the 5 percent-damped spectrum of the design basis earthquake for periods from 0.2T seconds to 1.5T seconds. Each pair of time histories shall be applied simultaneously to the model considering torsional effects.

The parameter of interest shall be calculated for each time-history analysis. If three time-history analyses are performed, then the maximum response of the parameter of interest shall be used for design. If seven or more time-history analyses are performed, then the average value of the response parameter of interest may be used for design.

UBC §1646.6.2 Elastic Time-History Analysis. Elastic time history shall conform to Sections 1646.1, 1646.2, 1646.3, 1646.5.2, 1646.5.4, 1646.5.5, 1646.5.6, 1646.5.7 and 1646.6.1. Response parameters from elastic time-history analysis shall be denoted as Elastic Response Parameters. All elements shall be designed using strength design. Elastic Response Parameters may be scaled in accordance with §1646.5.4.

UBC §1646.6.3 Nonlinear Time-History Analysis

UBC §1646.6.3.1 Nonlinear Time History. Nonlinear time-history analysis shall meet the requirements of §1644.10 and time histories shall be developed and results determined in accordance with the requirements of §1646.6.1. Capacities and characteristics of nonlinear elements shall be modeled consistent with test data or substantiated analysis, considering the Importance Factor. The maximum inelastic response displacement shall not be reduced and shall comply with §1645.10.

UBC §1646.6.3.2 Design Review. When nonlinear time history analysis is used to justify a structural design, a design review of the lateral force resisting system shall be performed by an independent engineering team including persons licensed in the appropriate disciplines and experienced in seismic analysis methods. The lateral force resisting system design review shall include, but not be limited to, the following:

1. Review the development of site-specific spectra and ground motion time histories.
2. Review of the preliminary design of the lateral force resisting system.
3. Review of the final design of the lateral force resisting system and all supporting analyses.

The Engineer of Record shall submit with the plans and calculations a statement by all members of the engineering team doing the review stating that the above review has been performed.

Modify: UBC §1630 *Revise as follows:*

UBC §1647 Lateral Force on Elements of Structures, Nonstructural Components and Equipment Supported by Structures

UBC §1647.1 General. Elements of structures and their attachments, permanent nonstructural components and their attachments, and the attachments for permanent equipment supported by a structure shall be designed to resist the total design seismic forces prescribed in §1647.2. Attachments for floor- or roof-mounted equipment weighing less than 400 pounds (181 kg), and furniture need not be designed.

Attachments shall include anchorages and required bracing. Friction resulting from gravity loads shall not be considered to provide resistance to seismic forces.

When the structural failure of the lateral force-resisting systems of nonrigid equipment would cause a life hazard, such systems shall be designed to resist the seismic forces prescribed in §1647.2.

When ~~allowable design stresses~~ permissible design strengths and other acceptance criteria are not contained in or referenced by this code, such criteria shall be obtained from approved national standards subject to the approval of the building official.

UBC §1647.2 Design for Total Lateral Force. The total design lateral seismic force, F_p, shall be determined from the following formula:

~~$F_p = Z I_p C_p W_p$~~ ~~(47-2)~~

$$F_p = 4.0 C_a I_p W_p \qquad (47\text{-}1)$$

~~The values of Z and I_p shall be the values used for the structure from Tables 16-I and 16-K.~~

~~The coefficient C_p is for elements and components and for rigid and rigidly supported equipment. Rigid or rigidly supported equipment is defined as having a fundamental period less than or equal to 0.06 second. Nonrigid or flexibly supported equipment is defined as a system having a fundamental period, including the equipment, greater than 0.06 second.~~

~~The lateral forces calculated for nonrigid or flexibly supported equipment supported by a structure and located above grade shall be determined considering the dynamic properties of both the equipment and the structure which supports it, but the value shall not be less than that listed in Table 16-O. In the absence of an analysis or empirical data, the value of C_p for nonrigid or flexibly supported equipment located above grade on a structure shall be taken as twice the value listed in Table 16-O, but need not exceed 2.0.~~

> ***Exception*:** Piping, ducting and conduit systems which are constructed of ductile materials and connections may use the values of C_p from Table 16-O.

~~The value of C_p for elements, components and equipment laterally self-supported at or below ground level may be two thirds of the value set forth in Table 16-O. However, the design lateral forces for an element or component or piece of equipment shall not be less than would be obtained by treating the item as an independent structure and using the provisions of §1632.~~

Alternatively, F_p may be calculated using the following formula:

$$F_p = \frac{a_p C_a I_p}{R_p}\left(1 + 3\frac{h_x}{h_r}\right)W_p \qquad (47\text{-}2)$$

Except that:

F_p shall not be less than $0.7C_aI_pW_p$, and need not be more than $4C_aI_pW_p$ (47-3)

where:

h_x = element or component attachment elevation with respect to grade. h_x shall not be taken less than 0.0.

h_r = structure roof elevation with respect to grade.

a_p = in-structure Component Amplification Factor that varies from 1.0 to 2.5

A value for a_p shall be selected from Table 16-O. Alternatively, this factor may be determined based on the dynamic properties or empirical data of the component and the structure that supports it. The value shall not be taken less than 1.0

R_p is the Component Response Modification Factor that shall be taken from Table 16-O, except that R_p for anchorages shall equal 1.5 for shallow chemical anchors, or shallow cast-in-place anchors, or when the component is constructed of nonductile materials. Shallow anchors are those with an embedment length-to-diameter ratio of less than 8. Where anchorage is constructed of nonductile materials or adhesive, R_p shall equal 1.0.

The design lateral forces determined using Formula (47-1) or (47-2) shall be distributed in proportion to the mass distribution of the element or component.

Forces determined using Formula (47-1) or (47-2) shall be used to design members and connections that transfer these forces to the seismic resisting systems. Members and connection design shall use the load combinations and factors specified in §1612.2 or 1612.3. The Reliability/Redundancy factor, ρ, may be taken equal to 1.0.

For applicable forces and component response modification factors in connectors for exterior panels and diaphragms, refer to Sections 1648.2.4, 1648.2.8 and 1648.2.9.

Forces shall be applied in the horizontal direction that results in the most critical loadings for design.

UBC §1647.3 Specifying Lateral Forces. Design specifications for equipment shall either specify the design lateral forces prescribed herein or reference these provisions.

UBC §1647.4 Relative Motion of Equipment Attachments. For equipment in Category 1 and 2 buildings as defined in Table 16-K, the lateral force design shall consider the effects of relative motion of the points of attachment to the structure ~~(drift)~~, using the drift based upon Δ_M.

UBC §1647.5 Alternative Designs. Where an approved national standard or approved physical test data provide a basis for the earthquake-resistant design of a particular type of equipment or other nonstructural component, such a standard or data may be accepted as a basis for design of the items with the following limitations:

1. These provisions shall provide minimum values for the design of the anchorage and the members and connections that transfer the forces to the seismic resisting system.
2. The force, F_p, and the overturning moment used in the design of the nonstructural component shall not be less than 80 percent of the values that would be obtained using these provisions.

Modify: UBC §1631 *Revise as follows:*

UBC §1648 Detailed Systems Design Requirements

UBC §1648.1 General. All structural framing systems shall comply with the requirements of §1644. Only the elements of the designated seismic force resisting system shall be used to resist design seismic forces. The individual components shall be designed to resist the prescribed design seismic forces acting on them. The components shall also comply with the specific requirements for the material contained in Chapters 19 through 23. In addition such framing systems and components shall comply with the detailed system design requirements contained in §1648.

All building components in Seismic Zones 2, 3, and 4 shall be designed to resist the effects of the seismic forces prescribed herein and the effects of gravity loadings from dead, floor live and snow loads.

Consideration shall be given to design for uplift effects caused by seismic loads. ~~For materials which use working stress procedures, dead loads shall be multiplied by 0.85 when used to reduce uplift.~~

In Seismic Zones 2, 3, and 4, provision shall be made for the effects of earthquake forces acting in a direction other than the principal axes in each of the following circumstances:

The structure has plan irregularity Type 5 as given in Table 16-M.

The structure has plan irregularity Type 1 as given in Table 16-M for both major axes.

A column of a structure forms part of two or more intersecting lateral force resisting systems.

> *Exception*: If the axial load in the column due to seismic forces acting in either direction is less than 20 percent of the column ~~allowable~~ axial load capacity.

The requirement that orthogonal effects be considered may be satisfied by designing such elements for 100 percent of the prescribed design seismic forces in one direction plus 30 percent of the prescribed design seismic forces in the perpendicular direction. The combination requiring the greater component strength shall be used for design. Alternatively, the effects of the two orthogonal directions may be combined on a square root of the sum of the squares (SRSS) basis. When the SRSS method of combining directional effects is used, each term computed shall be assigned the sign that will result in the most conservative result.

UBC §1648.2 Structural Framing Systems

UBC §1648.2.1 General. Four types of general building framing systems defined in §1644.6 are recognized in these provisions and shown in Table 16-N. Each type is subdivided by the types of vertical elements used to resist lateral seismic forces. Special framing requirements are given in this section and in Chapters 19 through 23.

UBC §1648.2.2 Detailing for Combinations of Systems. For components common to different structural systems, the more restrictive detailing requirements shall be used.

UBC §1648.2.3 Connections. Connections that resist design seismic forces shall be designed and detailed on the drawings.

UBC §1648.2.4 Deformation Compatibility. All structural framing elements and their connections, not required by design to be part of the lateral force resisting system, shall be designed and or detailed to be adequate to maintain support of design dead plus live loads when subjected to the expected deformations caused by seismic forces. P-Δ effects on such elements shall be ~~accounted for~~ considered. Expected deformations shall be ~~estimated~~ determined as the greater of ~~3(R_W/8 times the computed elastic deformations at design seismic forces~~ the Maximum Inelastic Response Displacement, Δ_M, considering P-Δ effects determined in accordance with §1945.9.2 or the deformation induced by a story drift of 0.0025 times the story height. When computing expected deformations, the stiffening effect of those elements not part of the lateral force-resisting system shall be neglected.

For elements not part of the lateral force resisting system, the forces induced by the expected deformation may be considered as ultimate or factored forces. When computing the forces induced by expected deformations, the restraining effect of adjoining rigid structural and nonstructural elements shall be considered and a rational value of member and restraint stiffness shall be used. Inelastic deformations of members and connections may be considered in the evaluation, provided the assumed calculated capacities are consistent with member and connection design and detailing.

For concrete and masonry elements that are part of the lateral force resisting system, the assumed flexural and shear stiffness properties shall not exceed one-half of the gross section properties unless a rational cracked-section analysis is performed. Additional deformations that may result from foundation flexibility and diaphragm deflections shall be considered. For concrete elements not part of the lateral force resisting system, see §1921.7.

UBC §1648.2.4.1 Adjoining Rigid Elements. Moment-resisting frames and shear walls may be enclosed by or adjoined by more rigid elements provided it can be shown that the participation or failure of the more rigid elements will not impair the vertical and lateral load-resisting ability of the gravity load and lateral force resisting systems. The effects of adjoining rigid elements shall be considered when assessing whether a structure shall be designated regular or irregular in Section 1644.5.1.

UBC §1648.2.4.2 Exterior Elements. Exterior nonbearing, nonshear wall panels or elements that are attached to or enclose the exterior shall be designed to resist the forces per Formula (47-1) or (47-2) and shall accommodate movements of the structure based on Δ_M and ~~resulting from lateral forces response to the design basis ground motion Δ_M or~~ temperature changes. Such elements shall be supported by means of cast-in-place concrete or by mechanical connections and fasteners in accordance with the following provisions:

1. Connections and panel joints shall allow for a relative movement between stories of not less than two times the story drift cause by wind, ~~3(R_w/8) times~~ the calculated ~~elastic~~ story drift based on Δ_M ~~caused by design seismic forces~~ or $^{1}/_{2}$ inch (13 mm), whichever is greater.

2. Connections to permit movement in the plane of the panel for story drift shall be sliding connections using slotted or oversize holes, connections that permit movement by bending of steel, or other connections providing equivalent sliding and ductility capacity.
3. Bodies of connections shall have sufficient ductility and rotation capacity so as to preclude fracture of the concrete or brittle failures at or near welds.
4. The body of the connection shall be designed for ~~one and one-third times~~ the force determined by Formula ~~(30-1)~~ (47-2), where $R_p = 3.0$ and $a_p = 1.0$.
5. All fasteners in the connecting system such as bolts, inserts, welds and dowels shall be designed for ~~four times~~ the forces determined by Formula ~~(30-1)~~ (47-2), where $R_p = 1.0$ and $a_p = 1.0$.
6. Fasteners embedded in concrete shall be attached to, or hooked around, reinforcing steel or otherwise terminated so as to effectively transfer forces to the reinforcing steel.

UBC §1648.2.5 Ties and Continuity. All parts of a structure shall be interconnected and the connections shall be capable of transmitting the seismic force induced by the parts being connected. As a minimum any smaller portion of the building shall be tied to the remainder of the building with elements having at least a strength to resist ~~Z/3~~ $0.5C_aI$ times the weight of the smaller portion.

A positive connection for resisting a horizontal force acting parallel to the member shall be provided for each beam, girder or truss. This force shall not be less than ~~Z/5~~ $0.3C_aI$ times the dead plus live load.

UBC §1648.2.6 Collector Elements. Collector elements shall be provided that are capable of transferring the seismic forces originating in other portions of the building to the element providing the resistance to those forces. Collector elements and their connections shall resist the forces determined in accordance with Formula (48-1). In addition, collector elements and their connections shall have the strength to resist the combined loads resulting from the Special Seismic Load Combinations of Section 1612.4. The quantity E_M need not exceed the maximum force that can be transferred to the collector by the Lateral Force resisting System. For collector elements designed using the Allowable Stress Design requirements of Chapter 23, the quantity E_M may be reduced by 30 percent and a one-third increase shall be permitted in allowable stress.

UBC §1648.2.7 Concrete Frames. Concrete frames required by design to be part of the lateral force resisting system shall conform to the following:

1. In Seismic Zones 3 and 4 they shall be special moment-resisting frames.
2. In Seismic Zone 2 they shall, as a minimum, be intermediate moment-resisting frames.

UBC §1648.2.8 Anchorage of Concrete or Masonry Walls. Concrete or masonry walls shall be anchored to all floors and roofs that provide out-of-plane lateral support of the wall. The anchorage shall provide a positive direct connection between the wall and floor or roof construction capable of resisting the larger of the horizontal forces specified in this section, and Sections 1611.4 and 1647. In addition, in Seismic Zones 3 and 4, diaphragm to wall anchorage using embedded straps shall have the straps attached to or hooked around the reinforcing steel or otherwise terminated so as to effectively transfer forces to the reinforcing steel. Requirements for developing anchorage forces in diaphragms are given in Section 1648.2.9 below. Diaphragm deformation shall be considered in the design of the supported walls.

UBC §1648.2.8.1 Out-of-plane wall anchorage to flexible diaphragms. This section shall apply in Seismic Zones 3 and 4 where flexible diaphragms, as defined in Section 1645.6, provide lateral support for the walls.

1. Elements of the wall anchorage system shall be designed for the forces specified in Section 1647 ~~using a C_p value of 1.2~~ where $R_p = 3.0$ and $a_p = 1.5$.

   In Seismic Zone 4, the value of F_p used for the design of the elements of the wall anchorage system shall not be less than ~~300 pounds per lineal foot (94.38 kN/m) of wall~~ 420 pounds per lineal foot of wall substituted for E.

   See Section 1611.4 for minimum design forces in other seismic zones.
2. When elements of the wall anchorage system are not loaded concentrically or are not perpendicular to the wall, the system shall be designed to resist all components of the forces induced by the eccentricity.
3. When pilasters are present in the wall, the anchorage force at the pilasters shall be calculated considering the additional load transferred from the wall panels to the pilasters. However, the minimum anchorage force at a floor or roof shall be that specified in Section 1648.2.8.1, Item 1.

~~4. Strength design using a load factor of 1.7 for E in lieu of 1.4 as specified in Section 1921.2.7 shall be used for design embedded elements in concrete.~~

5~~4.~~ The strength design forces for steel elements of the wall anchorage system shall be ~~1.7~~ 1.4 times the forces otherwise required by this section.

6~~5~~. The strength design force for wood elements of the wall anchorage system shall be 0.85 times the forces otherwise required by this section and these wood elements shall have a minimum actual net thickness of 2.5 inches (62 mm).

UBC §1648.2.9 Diaphragms

1. The deflection in the plane of the diaphragm shall not exceed the permissible deflection of the attached elements. Permissible deflection shall be that deflection that will permit the attached element to maintain its structural integrity under the individual loading and continue to support the prescribed loads.
2. Floor and roof diaphragms shall be designed to resist the design seismic forces determined in accordance with the following formula:

$$F_{px} = \frac{F_t + \sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px} \qquad (48\text{-}1)$$

The force F_{px} determined from Formula (48-1) need not exceed ~~$0.75ZIw_{px}$~~ $1.0C_aIw_{px}$, but shall not be less than ~~$0.35ZIw_{px}$~~ $0.5C_aIw_{px}$.

When the diaphragm is required to transfer ~~lateral~~ design seismic forces from the vertical resisting elements above the diaphragm to other vertical resisting elements below the diaphragm due to offset in the placement of the elements or to changes in stiffness in the vertical elements, these forces shall be added to those determined from Formula (48-1).

3. Design seismic forces for flexible diaphragms providing lateral supports for walls or frames of masonry or concrete, shall be ~~calculated using an R_w not to exceed 6.~~ determined using Formula (48-1) based on the load determined in accordance with Section 1645.2 using an R_dR_o not exceeding 4.
4. Diaphragms supporting concrete or masonry walls shall have continuous ties or struts between diaphragm chords to distribute the anchorage forces specified in Section 1648.2.8. Added chords of subdiaphragms may be used to form subdiaphragms to transmit the anchorage forces to the main continuous crossties. The maximum length to width ratio of the wood structural sub diaphragm shall be $2\frac{1}{2}$: 1.
5. Where wood diaphragms are used to laterally support concrete or masonry walls, the anchorage shall conform to Section 1648.2.8 above. In Seismic Zones 2, 3 and 4 anchorage shall not be accomplished by use of toenails or nails subject to withdrawal, nor shall wood ledgers or framing be used in cross-grain bending or cross-grain tension, and the continuous ties required by Item 4 above shall be in addition to the diaphragm sheathing.

~~6. Connections of diaphragms to the vertical elements and to collectors and connections of collectors to the vertical elements in structures in Seismic Zones 3 and 4, having a plan irregularity of Type 1, 2, 3, or 4 in Table 16-M, shall be designed for 125 percent of the design seismic forces otherwise required. without considering the one-third increase usually permitted in allowable stresses for elements resisting earthquake forces.~~

~~7~~ 6. In structures in Seismic Zones 3 and 4 having a plan irregularity of Type 2 in Table 16-M, diaphragm chords and drag members shall be designed considering independent movement of the projecting wings of the structure. Each of these diaphragm elements shall be designed for the more severe of the following two assumptions:

Motion of the projecting wings in the same direction.

Motion of the projecting wings in opposing directions.

> ***Exception*****:** This requirement may be deemed satisfied if the procedures of Section 1646 in conjunction with a three-dimensional model have been used to determine the lateral seismic forces for design.

UBC §1648.2.10 Framing Below the Base. The strength and stiffness of the framing between the base and the foundation shall not be less than that of the superstructure. The special detailing requirements of Chapters 19 and 22, as appropriate, shall apply to columns supporting discontinuous lateral force-resisting elements and to SMRF, IMRF, EBF, STMF and MMRWF system elements below the base that are required to transmit the forces resulting from lateral loads to the foundation.

UBC §1648.2.11 Building Separations. All structures shall be separated from adjoining structures. Separations shall allow for ~~3(R_w/8) times~~ the displacement ~~due to seismic forces~~ Δ_M. Adjacent buildings on the same property shall be separated by at least Δ_{MT} where

$$\Delta_{MT} = \sqrt{(\Delta_{M1})^2 + (\Delta_{M2})^2} \qquad (48\text{-}2)$$

and Δ_{M1} and Δ_{M2} are the displacements of the adjacent buildings.

When a structure adjoins a property line not common to a public way, that structure shall also be set back from the property line by at least ~~3(R_w/8) times~~ the displacement Δ_M of that structure.

> ***Exception*****:** Smaller separations or property line setbacks may be permitted when justified by rational analyses based on maximum expected ground motions. ~~As a minimum, building separations or property line setbacks shall not be less than (R_w/8) > = 1 times the sum of displacement due to code-specified forces.~~

Modify: **UBC §1632 and UBC §1633** *Revise as follows:*

UBC §1649 Nonbuilding Structures

UBC §1649.1 General

UBC §1649.1.1 Scope. Nonbuilding structures include all self-supporting structures other than buildings that carry gravity loads and resist the effects of earthquakes. Nonbuilding structures shall be designed to provide the strength required to resist the displacements induced by the minimum lateral forces specified in this section. Design shall conform to the applicable provisions of other sections as modified by the provisions contained in Section 1649.

UBC §1649.1.2 Criteria. The minimum design ~~lateral~~ seismic forces prescribed in this section are at a level that produce displacements in a fixed base, elastic model of the structure, comparable to those expected of the real structure when responding to the design basis ground motion. Reductions in these forces, using the coefficient R is permitted, where ~~at a service level (rather than yield or ultimate level).~~ the design of nonbuilding structures ~~shall~~ provides sufficient strength and ductility, consistent with the provisions specified herein for buildings, to resist the effects of seismic ground motions as represented by these design forces.

When applicable, ~~allowable stresses~~ design strength and other detailed design criteria shall be obtained from other sections or their referenced standards. The design of nonbuilding structures shall use the load combinations or factors specified in Section 1612.2 or 1612.3. For nonbuilding structures designed using Sections 1649.3, 1649.4 or 1649.5, the Reliability/Redundancy factor, ρ, may be taken as 1.0.

When applicable design ~~stresses~~ strengths and other design criteria are not contained in or referenced by this code, such criteria shall be obtained from approved national standards.

UBC §1649.1.3 Weight W. The weight, W, for nonbuilding structures shall include all dead loads as defined for buildings in Section 1645.1.1. For purposes of calculating design seismic forces in nonbuilding structures, W shall also include all normal operating contents for items such as tanks, vessels, bins and piping.

UBC §1649.1.4 Period. The fundamental period of the structure shall be determined by rational methods such as by using Method B in Section 1645.2.2.

UBC §1649.1.5 Drift. The drift limitations of Section 1645.10 need not apply to nonbuilding structures. Drift limitations shall be established for structural or nonstructural elements whose failure would cause life hazards. P-Δ effects shall be considered for structures whose calculated drifts exceed the values in Section 1645.1.3.

UBC §1649.1.6 Interaction Effects. In Seismic Zones 3 and 4, structures that support flexible nonstructural elements whose combined weight exceeds 25 percent of the weight of the structure shall be designed considering interaction effects between the structure and the supported elements.

UBC §1649.2 Lateral Force. Lateral force procedures for nonbuilding structures with structural systems similar to buildings (those with structural systems that are listed in Table 16-N) shall be selected in accordance with the provisions of Section 1644.

> *Exception:* Intermediate moment-resisting frames (IMRF) may be used in Seismic Zones 3 and 4 for nonbuilding structures in Occupancy Categories 3 and 4 if: 1) the structure is less than 50 feet (15 240 mm) in height, and 2) ~~an R_w = 40~~ the product $R_d R_o$ used in reducing calculated member forces and moments does not exceed 2.8. ~~is used for design~~.

UBC §1649.3 Rigid Structures. Rigid structures (those with period T less than 0.06 second), and ~~including~~ their anchorages, shall be designed for the lateral force obtained from Formula (49-1).

~~$V = 0.5ZIW$~~ ~~(32-1)~~

$$V = 0.7\, C_a I W \qquad (49\text{-}1)$$

The force V shall be distributed according to the distribution of mass and shall be assumed to act in any horizontal direction.

UBC §1649.4 Tanks with Supported Bottoms. Flat bottom tanks or other tanks with supported bottoms, founded at or below grade, shall be designed to resist the seismic forces calculated using the procedures in Section 1647 for rigid structures considering the entire weight of the tank and its contents. Alternatively, such tanks may be designed using one of the two procedures described below.

1. A response spectrum analysis, which includes consideration of the actual ground motion anticipated at the site and the inertial effects of the contained fluid.
2. A design basis prescribed for the particular type of tank by an approved national standard, provided that the seismic zones and occupancy categories shall be in conformance with the provisions of Section 1644.4 and 1644.2, respectively.

UBC §1649.5 Other Nonbuilding Structures. Nonbuilding structures not covered by Section 1649.3 and 1649.4 shall be designed to resist ~~minimum~~ design seismic-~~lateral~~ forces not less than those determined in accordance with the provisions of Section 1645 with the following additions and exceptions:

1. The factors R_0 and R_d ~~R_w~~ shall be as ~~given~~ set forth in Table 16-P. ~~The ratio C/ R_w used for design shall not be less than 0.4~~

   The total design base shear determined in accordance with Section 1645.2 shall not be less than the following:

   for Seismic Zone 4 $$V = 0.56 C_a I W \qquad (49\text{-}2a)$$

   In addition to Seismic Zone 4, the total base shear shall also not be less than the following:

   $$V = 1.6 \frac{Z N_v}{R} I W \qquad (49\text{-}2b)$$

2. The vertical distribution of the design ~~lateral~~ seismic forces in structures covered by this section may be determined by using the provisions of Section 1645.5 or by using the procedures of Section 1646.

   Exception: For irregular structures assigned to Occupancy Categories 1 and 2 that cannot be modeled as a single mass, the procedures of Section 1646 shall be used.

3. Where an approved national standard provides a basis for the earthquake-resistant design of a particular type of nonbuilding structure covered by this Section, 1649.5, such a standard may be used, subject to the limitations in this section:

The seismic zones and occupancy categories shall be in conformance with the provisions of Section 1644.4 and 1644.2, respectively.

The values for total lateral force and total base overturning moment used in design shall not be less than 80 percent of the values that would be obtained using these provisions.

UBC §1650 Earthquake Recording Instrumentations

UBC §1650.1 For earthquake recording instrumentations, see UBC Appendix Chapter 16, Division II.

Add new division: UBC Chapter 16 *Add a division as follows:*

Division VI Soil Profile Types

UBC §1662 Site Categorization Procedure

UBC §1662.1 Scope. This part describes the procedure for determining the soil profile Type S_A through S_F in accordance with Table 16-J.

UBC §1662.2 Definitions. Soil profile types are defined as follows:

S_A	Hard rock with measured shear wave velocity, $\bar{v}_s > 5{,}000$ ft./sec. (1500 m/s).
S_B	Rock with 2,500 ft./sec. $< \bar{v}_s \leq 5{,}000$ ft./sec. (760 m/s $< \bar{v}_s \leq$ 1500 m/s).
S_C	Very dense soil and soft rock with 1,200 ft./sec. $< \bar{v}_s \leq 2{,}500$ ft./sec. (360 m/s $< \bar{v}_s \leq$ 760 m/s) or with either $N > 50$ or $\bar{S}_u$ 2,000 psf (100 *kPa*).
S_D	Stiff soil with 600 ft./sec. $\leq \bar{v}_s \leq 1{,}200$ ft./sec. (180 m/s $\bar{v}_s \leq$ 360 m/s) or with $15 \leq N \leq 50$ or 1,000 psf $\leq \bar{S}_u \leq$ 2,000 psf (50 $kPa \leq \bar{S}_u \leq$ 100 kPa).
S_E	A soil profile with $\bar{v}_s \langle$ 600 ft./sec. (180 m/s) or any profile with more than 10 ft. (3048 mm) of soft clay defined as soil with $PI > 20$, w_{mc} 40 percent, and $\bar{S}_u <$ 1000 psf (50 *kPa*).
S_F	Soils requiring site-specific evaluation:

1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils.
2. Peats and/or highly organic clays [$H >$ 10 ft. (3048 mm) of peat and/or highly organic clay where H = thickness of soil]
3. Very high plasticity clays [$H >$ 25 ft. (7620 mm) with $PI > 75$]
4. Very thick soft/medium stiff clays [$H >$ 120 ft. (36 580 mm)]

***Exception*:** When the soil properties are not known in sufficient detail to determine the Soil Profile Type, Type S_D shall be used. Soil Profile Type S_E need not be assumed unless the building official determines that Soil Profile Type S_E may be present at the site or in the event that Type S_E is established by geotechnical data.

The criteria set forth in the definition for Soil Profile Type S_F requiring site-specific evaluation shall be considered. If the site corresponds to this criteria, the site shall be classified as Soil Profile Type S_F and a site-specific evaluation shall be conducted.

UBC §1662.2.1 $\bar{v}_s$ Average Shear Wave Velocity. $\bar{v}_s$ shall be determined in accordance with the following formula:

$$\bar{v}_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}} \qquad (62\text{-}1)$$

where:

d_i = Thickness of layer i in feet (m).

v_{si} = Shear wave velocity in layer i in ft./sec. (m/sec)

UBC §1662.2.2 N Average Field Standard Penetration Resistance and N_{CH} Average Standard Penetration Resistance for Cohesionless Soil Layers. $\bar{N}$ and $\bar{N}_{CH}$ shall be determined in accordance with the following formulae:

$$\bar{N} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}} \qquad (62\text{-}2)$$

and

$$\bar{N}_{CH} = \frac{d_s}{\sum_{i=1}^{n} \frac{d_i}{N_i}} \qquad (62\text{-}3)$$

where:

d_i = thickness of layer i in feet (mm).

d_s = total thickness of cohesionless soil layers in the top 100 ft. (30 480 mm).

N_i = standard penetration resistance of soil layer in accordance with approved nationally recognized standards.

UBC §1662.2.3 $\bar{S}_u$, Average Undrained Shear Strength. $\bar{S}_u$ shall be determined in accordance with the following formula:

$$\bar{S}_u = \frac{d_c}{\sum_{i=1}^{n} \frac{d_i}{S_{ui}}} \qquad (62\text{-}4)$$

where:

d_c = total thickness (100 - d_s) of cohesive soil layers in the top 100 ft. (30 480 mm).

S_{ui} = undrained shear strength in accordance with approved nationally recognized standards, not to exceed 5000 psf (250 *kPa*).

UBC §1662.2.4 Soft Clay Profile S_E. The existence of a total thickness of soft clay > 10 ft. (3048 mm) shall be investigated where a soft clay layer is defined by $\bar{S}_u <$ 500 psf (24 *kPa*), w_{mc} 40 percent, and $PI > 20$. If this criteria is met, the site shall be classified as Soil Profile Type S_E.

UBC §1662.3.5 Soil profiles S_C, S_D and S_E. Sites with Soil Types S_C, S_D and S_E shall be classified by using one of the following three methods with $\bar{v}_s$, $\bar{N}$ and $\bar{S}_u$ computed in all cases as specified in Section 1662.2.

a. $\bar{v}_s$ for the top 100 ft. (30 480 mm) ($\bar{v}_s$ method).

b. $\bar{N}$ for the top 100 ft. (30 480 mm) ($\bar{N}$ method).

c. $\bar{N}_{CH}$ for cohesionless soil layers ($PI < 20$) in the top 100 ft. (30 480 mm) and average $\bar{S}_u$ for cohesive soil layers ($PI > 20$) in the top 100 ft. (30 480 mm) ($\bar{S}_u$ method).

UBC §1662.3.6 Rock profiles, S_A and S_B. The shear wave velocity for rock, Soil Profile Type S_B, shall be either measured on site or estimated by a geotechnical engineer, engineering geologist or seismologist for competent rock with moderate fracturing and weathering. Softer and more highly fractured and weathered rock shall be either be measured on site for shear wave velocity or classified as Soil Profile Type S_C.

The hard rock, Soil Profile Type S_A, category shall be supported by shear wave velocity measurement either on site or on profiles of the same rock type in the same formation with an equal or greater degree of weathering and fracturing. Where hard rock conditions are known to be continuous to a depth of 100 ft. (30 480 mm), surficial shear wave velocity measurements may be extrapolated to assess $\bar{v}_s$. The rock categories, Soil Profile Type S_A and S_B shall not be used if there is more than 10 ft. (3048 mm) of soil between the rock surface and the bottom of the spread footing or mat foundation.

The definitions presented herein shall apply to the upper 100 ft. (30 480 mm) of the site profile. Profiles containing distinctly different soil layers shall be subdivided into those layers designated by a number from 1 to *n* at the bottom where there are a total of n distinct layers in the upper 100 ft. (30 480 mm). The symbol i then refers to any one of the layers between 1 and n.

The zone shall be determined from the seismic zone map in Figure 16-2.

(Note: No revisions are proposed to this table. Reprinted for reference.)

Table 16-I. *Seismic Zone Factor Z*

Zone	1	2A	2B	3	4
Z	0.075	0.15	0.20	0.30	0.40

Modify: **Table 16-J.** *Delete and substitute as follows:*

Table 16-J. *Soil Profile Types*

Soil Profile Type	Soil Profile Name/Generic Description	Average Soil Properties for Top 100 Feet (30 480 mm) of Soil Profile		
		Shear Wave Velocity, $\bar{v}_s$ feet/second (m/s)	Standard Penetration Test, $\bar{N}$ (or $\bar{N}_{CH}$ for cohesionless soil layers) (blows/foot)	Undrained Shear Strength, $\bar{S}_u$ psf (kPa)
S_A	Hard rock	> 5,000 (1500)		
S_B	Rock	2,500 to 5,000 (760 to 1500)		
S_C	Very dense soil and soft rock	1,200 to 2,500 (360 to 760)	> 50	> 2,000 (100)
S_D	Stiff soil profile	600 to 1,200 (180 to 360)	15 to 50	1,000 to 2,000 (50 to 100)
S_E[1]	Soft soil profile	< 600 (180)	< 15	< 1,000 (50)
S_F	Soil requiring site-specific evaluation. See Section 1644.3.1			

1. Soil profile Type S_E also includes any soil profile with more than 10 ft (3048 mm) of soft clay defined as a soil with a plasticity index, $PI > 20$, $w_{mc} > 40$ percent and $\bar{S}_u < 500$ psf (25 kPa). The Plasticity Index, PI, the moisture content, w_{mc}, shall be determined in accordance with approved national standards.

Table 16-K. Occupancy Category

(Table unchanged)

Modify: **Tables 16-L and 16-M.** *Revise as follows:*

Table 16-L. *Vertical Structural Irregularities*

Irregularity Type and Definition	**Reference Section**
1. *Stiffness irregularity - soft story* A soft story is one in which the lateral stiffness is less than 70 percent of that in the story above or less that 80 percent of the average stiffness of the three stories above.	~~1627.8.3, Item 2~~ 1644.8.4, Item 2
2. *Weight (mass) irregularity* Mass irregularity shall be considered to exist where the effective mass of any story is more than 150 percent of the effective mass of an adjacent story. A roof which is lighter than the floor below need not be considered.	~~1627.8.3, Item 2~~ 1644.8.4, Item 2
3. *Vertical geometric irregularity* Vertical geometric irregularity shall be considered to exist where the horizontal dimension of the lateral force-resisting system in any story is more than 130 percent of that in an adjacent story. One-story penthouses need not be considered.	~~1627.8.3, Item 2~~ 1644.8.4, Item 2
4. *In-plane discontinuity in vertical lateral force resisting element* An in-plane offset of the lateral load-resisting elements greater than the length of those elements.	~~1628.7~~ 1645.8
5. *Discontinuity in capacity - weak story* A weak story is one in which the story strength is less than 80 percent of that in the story above. The story strength is the total strength of all seismic resisting elements sharing the story shear for the direction under consideration.	~~1627.9.1~~ 1644.9.1

Table 16-M. *Plan Structural Irregularities*

Irregularity Type and Definition	**Reference Section**
1. *Torsional irregularity - to be considered when diaphragms are not flexible* Torsional irregularity shall be considered to exist when the maximum story drift computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts of the two ends of the structure	~~1631.2.9, Item 6~~ 1648.2.9, Item 6
2. *Reentrant corners* Plan configurations of a structure and its lateral force-resisting system contain reentrant corners, where both projections of the structure beyond a reentrant corner are greater than 15 percent of the plan dimension of the structure in the given direction.	~~1631.2.9, Items 6 and 7~~ 1648.2.9, Item 6
3. *Diaphragm discontinuity* Diaphragms with abrupt discontinuities or variations in stiffness, including those having cutout or open areas greater than 50 percent of the gross enclosed area of the diaphragm, or changes in effective diaphragm stiffness of more than 50 percent from one story to the next.	~~1631.2.9, Item 6~~ 1648.2.9, Item 6
4. *Out-of-plane offsets* Discontinuities in a lateral force path, such as out-of-plane offsets of the vertical elements.	~~1628.7;~~ ~~1631.2.9, Item 6; 2211.8~~ 1645.8; 2211.8
5. *Nonparallel systems* The vertical lateral load-resisting elements are not parallel to or symmetric about the major orthogonal axes of the lateral force-resisting system.	~~1631.1~~ 1648.1

Modify: **Table 16-N, 1996 Accumulative Supplement.** *Revise as follows:*

Table 16-N. *Structural Systems*[1]

Basic Structural System[2]	Lateral Force Resisting System Description	~~R_w~~1	R_d	R_o	Height Limit for Seismic Zones 3 and 4 X 304.8 for mm
1. Bearing wall system	1. Light-framed walls with shear panels				
	a. Wood structural panel walls for structures three stories or less	~~8~~	2.2	2.5	65
	b. All other light framed walls	~~6~~	1.8	2.5	65
	2. Shear walls				
	a. Concrete	~~6~~	1.8	2.5	160
	b. Masonry	~~6~~	1.8	2.5	160
	3. Light steel-framed bearing walls with tension-only bracing	4	1.4	2.0	65
	4. Braced frames where bracing carries gravity load				
	a. Steel	~~6~~	2.2	2.0	160
	b. Concrete[3]	4	1.4	2.0	—
	c. Heavy timber	4	1.4	2.0	65
2. Building frame system	1. Steel eccentrically braced frame (EBF)	~~10~~	2.8	2.5	240
	2. Light-framed walls with shear panels				
	a. Wood structural panel walls for structures three stories or less	~~9~~	2.6	2.5	65
	b. All other light framed walls	~~7~~	2.0	2.5	65
	3. Shear Walls				
	a. Concrete	~~8~~	2.2	2.5	240
	b. Masonry	~~8~~	2.2	2.5	160
	4. Ordinary braced frames				
	a. Steel	~~8~~	2.8	2.0	160
	b. Concrete[3]	~~8~~	2.8	2.0	—
	c. Heavy Timber	~~8~~	2.8	2.0	65
	5. Special concentrically braced frames				
	a. Steel	~~9~~	3.2	2.0	240

Table 16-N. (Continued) *Structural Systems*[1]

Basic Structural System[2]	Lateral Force Resisting System Description	~~R_w~~[1]	R_d	R_o	Height Limit for Seismic Zones 3 and 4 X 304.8 for mm
3. Moment-resisting frame system	1. Special moment resisting frame (SMRF)				
	a. Steel	~~12~~	3.4	2.5	N.L.
	b. Concrete	~~12~~	3.4	2.5	N.L.
	2. Masonry moment resisting wall frame	~~9~~	2.6	2.5	160
	3. Concrete intermediate moment resisting frame (IMRF)[4]	~~8~~	2.2	2.5	—
	4. Ordinary moment resisting frame (OMRF)				
	a. Steel[5]	~~6~~	1.8	2.5	160
	b. Concrete[6]	~~5~~	1.4	2.5	—
	5. Special truss moment frames of steel (STMF)	~~10~~	2.6	2.5	240
4. Dual systems	1. Shear Walls				
	a. Concrete with SMRF	~~12~~	3.4	2.5	N.L.
	b. Concrete with steel OMRF	~~6~~	1.7	2.5	160
	c. Concrete with concrete IMRF[4]	~~9~~	2.6	2.5	160
	d. Masonry with SMRF	~~8~~	2.2	2.5	160
	e. Masonry with steel OMRF	~~6~~	1.7	2.5	160
	f. Masonry with concrete IMRF[4]	~~7~~	1.7	2.5	—
	g. Masonry with Masonry MRWF		2.4	2.5	160
	2. Steel EBF				
	a. With steel SMRF	~~12~~	3.4	2.5	N.L.
	b. With steel OMRF	~~6~~	1.7	2.5	160
	3. Ordinary braced frames				
	a. Steel with steel SMRF	~~10~~	2.6	2.5	N.L.
	b. Steel with steel OMRF	~~6~~	1.7	2.5	160
	c. Concrete with concrete SMRF[3]	~~9~~	2.6	2.5	—
	d. Concrete with concrete IMRF[3]	~~6~~	1.7	2.5	—
	4. Special concentrically braced frames				
	a. Steel with steel SMRF	~~11~~	3.0	2.5	N.L.
	b. Steel with steel OMRF	~~6~~	1.7	2.5	160

Table 16-N. (Continued) *Structural Systems*[1]

Basic Structural System[2]	Lateral Force Resisting System Description	~~R_w~~1	R_d	R_o	Height Limit for Seismic Zones 3 and 4 X 304.8 for mm
5. Cantilevered column building systems	1. Cantilevered Column Elements	~~9~~	1.2	1.8	35[7]
6. Shear wall-frame interaction systems	1. Concrete[8]		2.2	2.5	
~~5.~~ 7. Undefined systems	See §1644.6.7 and §1644.9.2	—	—		—

N.L.- no limit

[1] ~~Basic structural systems are defined in Section 1644.6.~~ See §1645.4 for combination of structural systems.

[2] ~~See Section 1645.4 for combination of structural systems.~~ Basic structural systems are defined in §1644.6

[3] Prohibited in Seismic Zones 3 and 4.

[4] Prohibited in Seismic Zones 3 and 4, except as permitted in Section 1649.2.

[5] Ordinary moment-resisting frames in Seismic Zone 1 meeting the requirements of §2211.6 may use ~~an R_w value of 12~~ an R value of 8.

[6] Prohibited in Seismic Zones 2A, 2B, 3 and 4. See §1648.2.7.

[7] Total height of the building including cantilevered columns.

[8] Prohibited in Seismic Zones 2A, 2B, 3, and 4.

Replace: Table 16-O. *Delete existing table and substitute as follows:*

Table 16-o. Horizontal Force Factors, a_p and R_p

Elements of Structures and Nonstructural Components and Equipment[1]	**a_p**	**R_p**	**Footnote**
1. *Elements of Structures*			
a. Walls including the following:			
Unbraced (cantilevered) parapets	2.5	3.0	
Exterior walls at or above the ground floor and parapets braced above their centers of gravity	1.0	3.0	2
All interior bearing and nonbearing walls	1.0	3.0	2
b. Penthouse (except when framed by an extension of the structural frame)	2.5	4.0	
c. Connections for prefabricated structural elements other than walls. See also Section 1647.2	1.0	3.0	3
2. *Nonstructural Components*			
a. Exterior and interior ornamentations and appendages	2.5	3.0	
b. Chimneys, stacks and trussed towers supported on or projecting above the roof			
Laterally braced or anchored to the structural frame at a point below their centers of mass	2.5	3.0	
Laterally braced or anchored to the structural frame at or above the centers of mass	1.0	3.0	
c. Signs and billboards.	2.5	3.0	
d. Storage racks (include contents) over six feet (1829 mm) tall.	2.5	4.0	4
e. Permanent floor-supported cabinets and book stacks more than 6 feet (1829 mm) in height (include contents).	1.0	3.0	5
f. Anchorage and lateral bracing for suspended ceilings and light fixtures.	1.0	3.0	3, 6, 7, 10
g. Access floor systems.	1.0	3.0	4, 5, 9
h. Masonry or concrete fences over 6 feet (1829 mm) high.	1.0	3.0	
i. Partitions.	1.0	3.0	

Table 16-o. (Continued) Horizontal Force Factors, a_p and R_p

Elements of Structures and Nonstructural Components and Equipment[1]	a_p	R_p	Footnote
3. Equipment			
a. Tanks and vessels (include contents), including support systems	1.0	3.0	
b. Electrical, mechanical and plumbing equipment and associated conduit, and ductwork and piping.	1.0	3.0	5, 8, 11, 12, 13, 14, 15, 16
c. Any equipment laterally braced or anchored to the structural frame at a point below their center of mass.	2.5	3.0	5, 10, 14, 15, 16
d. Anchorage of emergency power supply systems and essential communications equipment. Anchorage and support systems for battery racks and fuel tanks necessary for operation of emergency equipment. See also Section 1647.2	1.0	3.0	17, 18
e. Temporary containers with flammable or hazardous materials.	1.0	3.0	19
4. Other Components			
a. Rigid components with ductile material and attachments.	1.0	3.0	1
b. Rigid components with nonductile material or attachments.	1.0	1.5	1
c. Flexible components with ductile material and attachments.	2.5	3.0	1
d. Flexible components with nonductile material or attachments.	2.5	1.5	1

[1] See Section 1642 for definitions of flexible components and rigid components.

[2] See Section 1648.2.4 and 1648.2.8 for concrete and masonry walls and Section 1647.2 for connections for panel connectors for panels.

[3] Applies to Seismic Zones Nos. 2, 3, and 4 only.

[4] Ground supported steel storage racks may be designed using the provisions of Section 1649. Chapter 22, Division VI may be used for design provided seismic design forces are equal to or greater than those specified in Sections 1647.2 or 1649.2, as appropriate.

[5] Only anchorage or restraints need be designed.

[6] Ceiling weight shall include all light fixtures and other equipment or partitions which are laterally supported by the ceiling. For purposes of determining the seismic force, a ceiling weight of not less than 4 pounds per square foot (19.5 kh/m^2) shall be used.

[7] Ceilings constructed of lath and plaster or gypsum board screw or nail attached to suspended members that support a ceiling at one level extending from wall to wall need not be analyzed provided the walls are not over 50 feet (15 m) apart.

[8] Equipment includes, but is not limited to, boilers, chillers, heat exchangers, pumps, air-handling units, cooling towers, control panels, motors, switchgear, transformers, and life-safety equipment. It shall include major conduit, ducting, and piping which services such machinery and equipment and fire sprinkler systems. See Section 1647.2 for additional requirements for determining a_p for nonrigid or flexibly mounted equipment.

[9] W_p for access floor systems shall be the dead load of the access floor system plus 25 percent of the floor live load plus a 10 psf (0.5 kP_a) partition load allowance.

[10] Light fixtures and mechanical services installed in metal suspension systems for acoustical tile and lay-in panel ceilings shall be independently supported from the structure above as specified in U.B.C. Standard 25-2, Part III.

[11] Seismic restraints may be omitted from piping and duct supports if all of the following conditions are satisfied:

  [11.1] Lateral motion of the piping or duct will not cause damaging impact with other systems.

  [11.2] The piping or duct is made of ductile material with ductile connections.

  [11.3] Lateral motion of the piping or duct does not cause impact of fragile appurtenances (e.g. sprinkler heads) with any other equipment, piping, or structural member.

  [11.4] Lateral motion of the piping or duct does not cause loss of system vertical support.

  [11.5] Rod-hung supports of less than 12 inches (30 mm) in length have top connections that cannot develop moments.

  [11.6] Support members cantilevered up from the floor are checked for stability.

[12] Seismic restraints may be omitted from electrical raceways such as cable trays, conduit, and bus ducts if all of the following conditions are satisfied:

  [12.1] Lateral motion of the raceway will not cause damaging impact with other systems.

  [12.2] Lateral motion of the raceway does not cause loss of system vertical support.

  [12.3] Rod-hung supports of less than 12 inches (30 mm) in length have top connections that cannot develop moments.

  [12.4] Support members cantilevered up from the floor are checked for stability.

[13] Piping, ducts, and electrical raceways that must be functional following an earthquake which span between different buildings or structural systems shall be sufficiently flexible to withstand relative motion of support points assuming out-of-phase motions

[14] Vibration isolators supporting equipment shall be designed for lateral loads or restrained from displacing laterally by other means. Restraint shall also be provided which limits vertical displacement, such that lateral restraints do not become disengaged. a_p and R_p for equipment supported on vibration isolators shall be taken as 2.5 and 1.5, respectively, except that if the isolation mounting

frame is supported by shallow or expansion anchors the design force for the anchors calculated by Equations 47-1, 47-2 or 47-3 shall be additionally multiplied by a factor of 2.0.

[15] Equipment anchorage shall not be designed such that lateral loads are resisted by gravity friction (e.g. friction clips).

[16] Expansion anchors which are required to resist seismic loads in tension shall not be used where operational vibrating loads are present.

[17] Movement of components within electrical cabinets, rack- and skid-mounted equipment, and portions of skid-mounted electromechanical equipment that may cause damage to other components by displacing, shall be restricted by attachment to anchored equipment or support frames.

[18] Batteries on racks shall be restrained against movement in all directions due to earthquake forces.

[19] Seismic restraints may include straps, chains, bolts, barriers, or other mechanisms which prevent sliding, falling, and breach of containment of flammable and toxic materials. Friction forces may not be used to resist lateral loads in these restraints unless positive uplift restraint is provided which ensures that the friction forces act continuously.

Modify: **Table 16-P.** *Revise as follows:*

Table 16-P. *~~R_w~~ R_o and R_d Factors for Nonbuilding Structures*

Structure Type	~~R_w~~	R_d	R_o
1. Vessels, including tanks and pressurized spheres on braced or unbraced legs.	~~3~~	1.2	1.8
2. Cast-in-place concrete silos and chimneys having walls continuous to the foundations.	~~5~~	2.0	1.8
3. Distributed mass cantilever structures such as stacks, chimneys, silos and skirt-supported vertical vessels.	~~4~~	1.6	1.8
4. Trussed towers (freestanding or guyed), guyed stacks and chimneys.	~~4~~	1.6	1.8
5. ~~Inverted pendulum~~ Cantilevered columns type structures.	~~3~~	1.2	1.8
6. Cooling towers.	~~5~~	2.0	1.8
7. Bins and hoppers on braced or unbraced legs.	~~4~~	1.6	1.8
8. Storage racks.	~~5~~	2.0	1.8
9. Signs and billboards.	~~5~~	2.0	1.8
10. Amusement structures and monuments.	~~3~~	1.2	1.8
11. All other self-supporting structures not otherwise covered.	~~4~~	1.6	1.8

Add tables: **Chapter 16, Division III.** *Add five new tables as follows:*

Table 16-Q. *Seismic Coefficient C_a*

Soil Profile Type	**Seismic Zone Factor, Z**				
	Z=0.075	**Z = 0.15**	**Z = 0.2**	**Z = 0.3**	**Z = 0.4**
S_A	0.06	0.12	0.16	0.24	$0.32N_a$
S_B	0.08	0.15	0.20	0.30	$0.40N_a$
S_C	0.09	0.18	0.24	0.33	$0.40N_a$
S_D	0.12	0.22	0.28	0.36	$0.44N_a$
S_E	0.19	0.30	0.34	0.36	$0.36N_a$
S_F	See Footnote 1				

[1] Site-specific geotechnical investigation and dynamic site response analysis shall be performed to determine seismic coefficients for Soil Profile Type S_F.

Table 16-R. *Seismic Coefficient C_v*

Soil Profile Type	**Seismic Zone Factor, Z**				
	Z=0.075	**Z = 0.15**	**Z = 0.2**	**Z = 0.3**	**Z = 0.4**
S_A	0.06	0.12	0.16	0.24	$0.32N_v$
S_B	0.08	0.15	0.20	0.30	$0.40N_v$
S_C	0.13	0.25	0.32	0.45	$0.56N_v$
S_D	0.18	0.32	0.40	0.54	$0.64N_v$
S_E	0.26	0.50	0.64	0.84	$0.96N_v$
S_F	See Footnote 1				

[1] Site-specific geotechnical investigation and dynamic site response analysis shall be performed to determine seismic coefficients for Soil Profile Type S_F.

Table 16-S. *Near Source Factor* N_a[1]

Seismic Source Type	Closest Distance to Known Seismic Source[2,3]		
	≤ 2 km	5 km	10 km
A	1.5	1.2	1.0
B	1.3	1.0	1.0
C	1.0	1.0	1.0

[1] The near-source factor may be based on the linear interpolation of values for distances other than those shown in the table.

[2] The location and type of seismic sources to be used for design shall be established based on approved geotechnical data (e.g. most recent mapping of active faults by the United States Geological Survey or the California Division of Mines and Geology).

[3] The closest distance to seismic source shall be taken as the minimum distance between the site and the area described by the vertical projection of the source on the surface (i.e., surface projection of fault plane). The surface projection need not include portions of the source at depths of 10 km, or greater. The largest value of the near-source factor considering all sources shall be used for design.

Table 16-T. *Near Source Factor* N_v[1]

Seismic Source Type	Closest Distance to Known Seismic Source[2,3]			
	≤ 2 km	5 km	10 km	15 km
A	2.0	1.6	1.2	1.0
B	1.6	1.2	1.0	1.0
C	1.0	1.0	1.0	1.0

[1] The near-source factor may be based on the linear interpolation of values for distances other than those shown in the table.

[2] The location and type of seismic sources to be used for design shall be established based on approved geotechnical data (e.g. most recent mapping of active faults by the United States Geological Survey or the California Division of Mines and Geology).

[3] The closest distance to seismic source shall be taken as the minimum distance between the site and the area described by the vertical projection of the source on the surface (i.e., surface projection of fault plane). The surface projection need not include portions of the source at depths of 10 km, or greater. The largest value of the near-source factor considering all sources shall be used for design.

Table 16-U. Seismic Source Type[1]

Seismic Source Type	Seismic Source Description	Seismic Source Definition	
		Maximum Moment Magnitude, M	Slip Rate, SR (mm/year)
A	Faults that are capable of producing large magnitude events and which have a high rate of seismic activity	M ≥ 7.0 and	SR ≥ 5
B	All faults other than Types A and C		
C	Faults which are not capable of producing large magnitude earthquakes and which have a relatively low rate of seismic activity	M < 6.5 and	SR ≤ 2

[1] Subduction sources shall be evaluated on a site specific basis.

Delete: **Figure 16-3 and substitute as follows:**

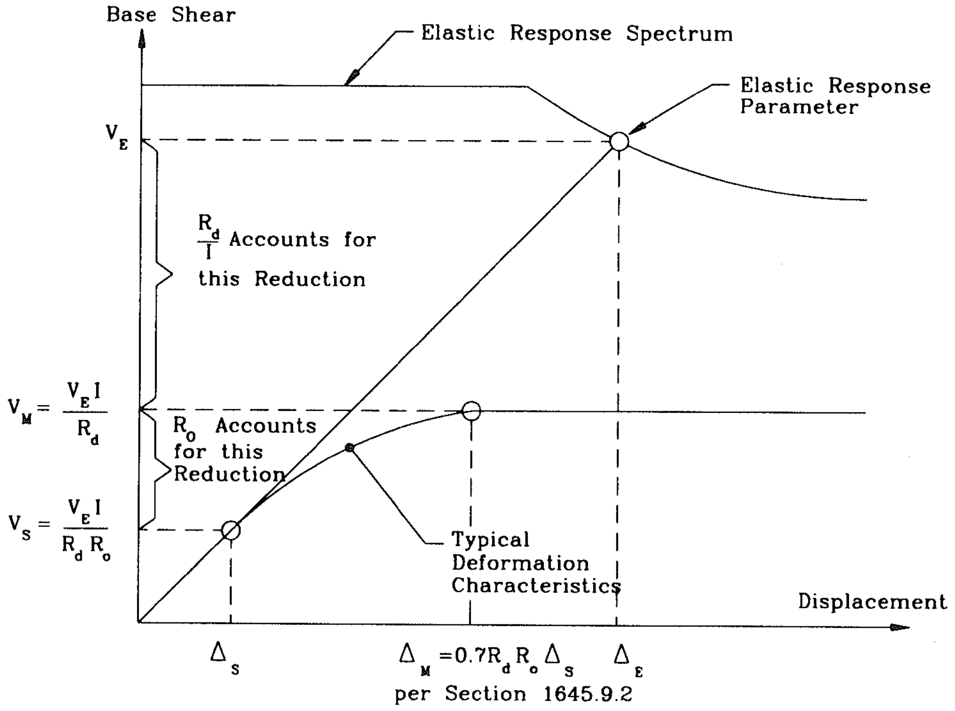

Base Shear
Elastic Response Spectrum
Elastic Response Parameter
V_E
$\frac{R_d}{I}$ Accounts for this Reduction
$V_M = \frac{V_E I}{R_d}$
R_0 Accounts for this Reduction
$V_S = \frac{V_E I}{R_d R_o}$
Typical Deformation Characteristics
Displacement
Δ_S
$\Delta_M = 0.7 R_d R_o \Delta_S$
Δ_E
per Section 1645.9.2

Figure 16-3. Relationship between base shear and displacement

Add: **Add Figure 16-4 as follows:**

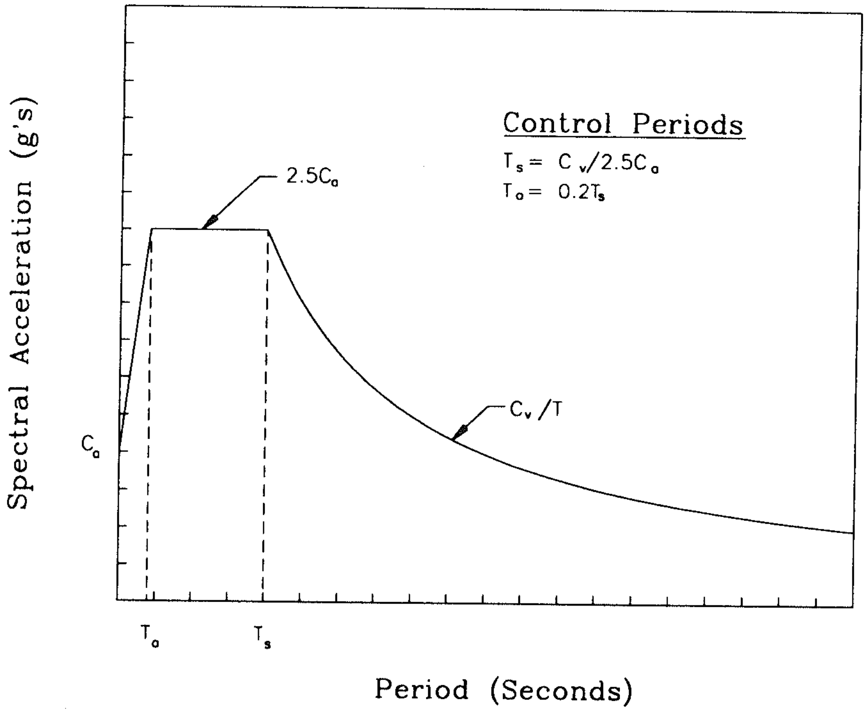


Figure 16-4. *Design response spectra*

APPENDIX C

SEAOC Strength Design Code Change Proposal

Commentary

General

The Strength Design Recommendations (Appendix C) represent the most significant development of the General Requirements in many years. The basic conversion from working stress to strength design is consistent with the direction of all material code requirements in the US and with the NEHRP seismic provisions, which serve as the source document for building codes other than the UBC. These Recommendations, along with a similar set of recommendations under development for the 1997 NEHRP, are intended to bring more consistency to seismic requirements nationally. A similar proposal has been submitted by the SEAOC Seismology to the ICBO for the 1997 UBC. It differs in the split R factors and differences in the definition of *R*. However, at the time of this Blue Book printing, the status of the ICBO proposal is uncertain.

Along with the basic change from working stress to strength design several conceptual changes have been introduced which reflect recent advancements in the field. These include the recognition of near-source effects in Seismic Zone 4, the incorporation of a redundancy factor which penalizes nonredundant lateral systems, the separation of the R-factor into ductility and overstrength components, and the adoption of a new set of soil profile categories. The soil profiles are considered in conjunction with seismic zone factors and near field factors (Zone 4) to produce site-dependant ground motion coefficients which define seismic response throughout the spectral range. Other major changes are made in the design requirements for lateral forces on elements of structures and nonstructural components and for nonbuilding structures.

The intent of the Seismology Committee is to provide exposure to these recommendations through this appendix prior to their adoption into building code requirements. Comments of the users of the recommendations are welcome and should be addressed to the Seismology Committee through the SEAOC office. It is intended that following further testing and refinements, these recommendations will be incorporated into the Chapter 1 Requirements in the body of the Blue Book.

UBC §1612 Combinations of Loads

The fundamental change from allowable stress design to strength basis is reflected in the load combinations given in UBC §1612. Subsection 1612.1 gives the basic factored load combinations that are intended for Strength Design or Load and Resistance Factor Design. These are consistent with both ASCE-7 and the 1994 (and proposed 1997) NEHRP Provisions. The load factor of 1.0 on the earthquake load, E, reflects the fact that the specified forces in the chapter are at design levels, without further amplification. They are typically about 1.4 times as high as current provisions, which essentially provides parity. To accommodate the preferences of engineers who wish to continue to work at allowable stress levels, load combinations are given in UBC §1612.3 for Allowable Stress Design.

The earthquake load, E, is a function of both horizontal and vertical components of ground motion (E_h and E_v). This is reflected in the earthquake loads given in UBC §1645.1.1, Formula 45-1. The horizontal component is taken from the base shear, V (UBC §1645.2). A small vertical component, E_v, is introduced into the earthquake load, E, for the Strength Design load combinations only. This component in seismic Zone 4 results in roughly $.2D$, which when added to $1.2D$ results in parity with the $1.4D$ factor in the 1994 UBC strength provisions (i.e. for concrete or masonry). In the allowable stress load combinations the vertical component is indirectly addressed through the more conservative dead and live load combination, so parity is more or less achieved.

The Special Seismic Load Combinations given in UBC §1612.4 are intended to cover conditions where ductility is lacking. The parameter E_m is introduced to represent the maximum earthquake force that can be developed in the structure. E_m exceeds the Design Base Shear by a factor of Ω_o, which is given as $1.1R_o$, a representation of the structural overstrength. This is equivalent to the $3/8R_w$ factor that appears in the current edition as a design requirement to address such nonductile issues as columns supporting discontinuous shear walls. For example, the average R_w value of 8 represents a three-times overstrength. This is about 1.25 times the overstrength represented by $1.1R_o$ using an equivalent R_o value of 2.4.

UBC §1644 Criteria Selection

UBC §1644.8.1 Basis for Design. Allowable Stress Design is retained for the design and evaluation of the foundation configuration at the soil-foundation interface for sliding and overturning moment resistance. Designs based on ASD have performed well during past strong ground motion earthquakes with adequate control on sliding and overturning deformation effects. With the option to use ASD the allowable soil capacities in UBC §1809.2 can be used directly, whereas Strength

Design would require a site specific study be made to establish the strength capacity of the soil for seismic loading.

For the Strength Design of the foundation footing elements, the soil pressures resulting from the ASD loads (D, L and E/1.4) in UBC §1612.3.1 may be load factored as per UBC §1612.2.1 for the evaluation of the Strength Design load actions on the foundation element sections.

UBC §1644.8.2 Simplified Static. The Simplified Static Procedure uses conservative base shear formulas that envelope the general building code requirements. Since the lateral force requirements have become more complicated, the committee felt that the casual or infrequent user of the requirements should have a procedure that can be quickly and conservatively used without a full application of the Static Force Procedure.

Use of the Simplified Static Procedure is limited to single family two story or less structures, light frame structures up to three stories in height, and any regular building up to two stories in height if approved by the Building Official. It is intended that the Simplified Static Procedure will result in a safe design with significantly less effort in the calculation of the required lateral forces for these building types.

UBC §1645 Minimum Design Lateral Forces and Related Effects

UBC §1645.1.1 Earthquake Loads

Redundancy/Reliability Factor. Redundancy is a characteristic of structures in which multiple paths of resistance to loads are provided. It has long been recognized as a desirable attribute, especially in cases where loads are likely to exceed service levels, as with earthquakes. While past commentaries to the Blue Book strongly recommended that the lateral load resisting system be made as redundant as possible, nothing was included to encourage this or give any guidance as to what constitutes a redundant structure.

The design philosophy implicit in the Blue Book assumes inelastic action of structures during a major seismic event. In order to prevent collapse of structures undergoing inelastic deformations it is essential to limit strength degradation. An important part of limiting the strength degradation is to provide multiple paths for the lateral loads. As portions of the structure yield, increases in loading may be distributed to other resisting elements and the energy absorbing inelastic action is distributed through the structure. Redundancy also decreases the probability that flaws in construction or design will have catastrophic results. Nonredundant systems are those for which failure of a small number of lateral force resisting components would result in the formation of a collapse mechanism for the structure.

Past earthquakes have shown repeatedly that redundant structures perform better than structures with few lateral load resisting elements. The recent Northridge event was no exception as findings by the SAC joint venture have indicated. Recently however, economic pressures have encouraged engineers and owners to limit the numbers of lateral load resisting elements in structures. The Redundancy Factor is intended to encourage engineers to design redundant structures. If the engineer is not able to achieve a redundant design, an increase in the seismic design loads are required in an attempt to reduce the magnitude of the inelastic response and therefore the ductility demand. The maximum value of ρ need not be taken greater than 1.5, however it is strongly recommended that the calculated value of ρ be as small as practical, preferably 1.0. Where ρ is calculated to be greater than 1.5, inelastic response capacities, quality control for both design and construction, and careful detailing of the structure should receive special consideration.

The initial drafts of the redundancy factor provision were based on the number of lateral load resisting elements provided and the plan area of the building. This did not address cases in which there was a large disparity in the magnitude of the lateral loads resisted by the elements. For these cases the beneficial effects of redundancy due to the number of elements provided would not be fully realized. This prompted the evolution of the redundancy factor to its present form in which not only the number of elements and plan area, but also the initial distribution of lateral load is accounted for.

The factor r_i is defined as the largest element-story shear ratio in a given story in a given direction. For example, a building with a story that has four columns that are all equal participants in the lateral force resisting system would have an $r_i = 0.25$. This approach was primarily intended to address buildings with torsional irregularities or with poor layouts of lateral force resisting elements. In such cases a linear static analysis will produce a high r value, resulting in a factor greater than 1.0. It is intended that the r calculation include the effects of actual and accidental torsion.

The value of r_{max} is determined as the largest of the values of r_i calculated for the lower two-thirds height of the building. This will allow engineers to continue the practice of deleting lateral load resisting elements at the upper levels without incurring an increase in design loads.

Specific refinements in the redundancy factor relating to Special Moment Resisting Frame structures were made. Moment frame design is typically controlled by drift criteria and since r does not affect the drift calculation, current design practice would not be affected by the required increase in lateral forces. For this reason an upper limit of 1.25 for the value of r was established. This will result in a minimum number of lateral load resisting elements when designing Special Moment Resisting Frames. For moment frames with multiple bays, 70 percent of column shear in any two adjacent bays is used to determine r_i. Thus the bay with the largest total shear is used to determine r_i.

For braced frames the value of r_i is determined by the lateral load resisting brace element with the largest horizontal force component.

For shear wall systems, the wall with the largest shear per foot is used to determine the value of r_i. The total lateral load in the wall is multiplied by $10/l_w$ and divided by the story shear to give a result that has the units of pounds/feet. All walls and wall piers designed as part of the lateral force resisting system are considered regardless of length. For this formulation, buildings with longer wall piers, all other things being equal, will be considered more redundant.

For dual systems all lateral load resisting elements are used in the determination of the redundancy factor. Where stiff elements are combined with flexible elements the value of will be somewhat more conservative than the case where all elements have similar stiffnesses. Because dual systems are inherently redundant, the redundancy factor may be taken as 80 percent of the value calculated by Formula 45-3.

The Seismology Committee has developed the redundancy factor so that the resulting ρ values are generally considered to be quite liberal. This was done because the concept of regulating redundancy is new and not extensively tested. Therefore, the factor should not at this point be considered a guideline for defining a *redundant* building, but rather a lower bound, below which a penalty is applied to the design force level.

The provision represents a consensus of the SEAOC Seismology Committee and is based on the cumulative judgment and experience of the engineers on that committee and in the local SEAOC sections. The provision matches very closely that which has been developed for inclusion into the 1997 NEHRP document. Research into quantifying redundancy has begun through the as yet unpublished ATC-19 and ATC-34 projects. The redundancy provision and the formula for ρ have been developed so that as more is learned about the effects of redundancy on the performance of structures, revisions can be made without significantly altering the form of the provision or the formula.

UBC §1645.1.3 P-delta Effects. Provisions from this section remain unchanged from the previous edition. Recent research (Bernal, 1991) has shown that p-delta has very little effect on structural response until dynamic instability is approached. It has also indicated that elastic stiffness is a poor measure of a structure's safety against instability. No changes to current p-delta provisions are proposed until a better approach for assessment of potential instability can be developed.

UBC §1645.2.3 Simplified Design Base Shear. The base shear is determined from Formula 45-11, which has a single variable, C_a. The value of C_a is obtained from Table 16-Q and is a function of both the seismic zone factor, Z, and the soil profile type. In place of determining the soil type, the simplified provisions allow the use of soil type S_D in Zones 3 and 4 and type S_E in Zones 1 and 2. In effect, this means that there is only one variable other than the weight of the structure required for the simplified static base shear. This variable is Z, the seismic zone. In addition, the provisions allow the use of a uniform distribution of lateral force over the height of the structure, unlike the triangular distribution required by the Static Force Procedure. This results in overturning moments that are about 17 percent less than the triangular distribution; however, the general conservatism in the Simplified Design Base Shear outweighs this effect.

This simplified procedure does not require detailed analysis, period calculations, drift calculations, and dynamic analysis. In general, the procedure results in forces approximately 20 to 35 percent higher than the forced calculated by the Static Force Procedure. The more frequent user of the code may choose to use the more complex Static Force Procedure; however, it is felt that the casual or occasional user would be willing to accept the increase in design forces for a simpler code formulation.

UBC §1645.3 Determination of Seismic Factors

UBC §1645.3.1 Determination of Ω_o. In previous versions of the recommendations, overstrength effects were approximated by the $3(R_w/8)$ factor. For example, in UBC-94, UBC §2211.5.1, checked using load combinations including gravity loads and the $3(R_w/8)E$ factor. Numerous similar requirements are found throughout the structural seismic Provisions. Although the $3(R_w/8)$ factor provides a reasonable approximation of the actual forces present in an inelastically responding structure, the implementation was cumbersome and offered little insight to the practicing engineer into the basis of the $3(R_w/8)$ "Overstrength" factor.

Use of the Overstrength Provisions. Specific provisions in Chapter 16 and in the material chapters require strength checks for structural elements using the *"Special Seismic Load Combinations"* of UBC §1612.4. These equations combine factored gravity loads with the quantity E_m defined as $E_m = \Omega_o E_h$, where E_h is the earthquake load due the lateral base shear. When allowable stress design is used, the quantity E_h is *not* reduced by 1.4. Provisions that invoke UBC §1612.4 typically will provide a method by which ASD member allowables can be factored up to strength levels. The engineer is in both cases presented with the estimated maximum gravity plus seismic load in the element and is required to show that the member in question has the strength to resist the imposed loading.

The *Special Seismic Load Combinations* utilizing the overstrength factor are needed to establish design loads for specific elements of the structure that are expected or required to remain elastic during the design basis ground motion; for example a discontinuous shear wall that bears on columns. 1994 UBC §1628.7.2 requires that the base columns be designed for $3(R_w 8)E$ axial loads. This version requires the column be designed for $\Omega_o E$ level axial loads. It is the intent that elements supporting the discontinuous system have *the strength to resist the maximum load that can develop* in the system. The actual lateral load that can be developed in the wall is approximated by the value $\Omega_o E$.

In Equation 45-13, $\Omega_o = 1.1\,R_o$ yields overstrength values ranging from about 2.2 to 3.5, depending on the value of R_o. In the current provisions, the $3(R_w/8)$ factor yields values ranging from 2.25 (R_W=6) to 4.5 (R_W=12). To compare the proposed provisions to the current provisions, the $3(R_W/8)$ factors may be divided by 1.4, yielded values ranging from 1.6 (R_W=6) to 3.2 (R_W=12), as compared to 2.2 and 3.5 in the proposed provisions for similar systems. Thus, the newly termed overstrength factor has nearly the same effect and impact as the previous $3(R_W/8)$ factor.

Future Overstrength Provisions. In the future, these provisions may allow the explicit determination of Ω_o by calculation, as opposed to the prescriptive values contained in this version. The procedure used to calculate overstrength may be established as follows:

Determination of Ω_o. The lateral strength, V_M, is determined by inelastic analysis. In this calculation, probable material overstrength and strain hardening effects should be included in establishing the member strengths. Also, to determine an upper bound on the lateral strength, live loads less than required by design would be used. These two factors would produce a lateral strength greater than the strength determined as described in the procedure for the determination of R_O below. The overstrength factor, Ω_o is then established as the ratio of the upper bound lateral strength to the strength level design base shear.

In these provisions, Ω_o is used to estimate the maximum loads in individual elements of the structure. However, a global value of Ω_o determined by inelastic analysis may be overly conservative for distinct elements. The above procedure can be expanded to permit peak forces in individual elements to be extracted directly from the inelastic analysis. For example, column axial loads in a frame building could be taken directly from the results of the inelastic analysis, rather than be factoring the design seismic forces by Ω_o.

Similarly, future provisions may permit the explicit calculation of the R_O factor, which would be established in a similar fashion:

Determination of R_O. The lateral strength of the structure, V_M, is determined by inelastic analysis and is defined as the base shear developed by the complete lateral system at the maximum inelastic displacement, Δ_M. In this calculation, specified material strengths are used, full factored gravity loads ($1.2D + 0.5L$) are used, and the distribution of load over height is taken as the normal triangular distribution. R_O is then calculated as the ratio of V_M to the design seismic forces, V_S, used to proportion the system. In the process, the engineer guesses V_S, designs the structure, computes V_M and determines R_O. The acceptance criteria is that the ratio of V_E/R_O should be less than or equal to the tabulated values of R_d. See Figure 16-3.

In both cases, the analysis must confirm that the yielding elements of the structure can sustain the inelastic deformations needed to attain the maximum inelastic response displacement, Δ_M. Otherwise, the lateral strength computed is not valid. In other words, the lateral strength, V_M, must be consistent with the deformational capacity of each yielding element. Therefore, while plastic analysis techniques can establish the lateral strength of the structure, they cannot establish the inelastic deformations present at the maximum strength and other methods must be employed.

Finally, the difference between R_O and Ω_o is as follows: R_O reflects the lower bound strength of the lateral system and is appropriate for use in establishing safe design base shears. Ω_o reflects the upper bound lateral strength and is appropriate for use in estimating the maximum forces developed in *nonyielding* elements of the lateral system during the design basis ground motion.

UBC §1645.9 Drift. In the current version of the Blue Book, structural displacements or drifts are computed using "Working Stress" level lateral forces. These displacements may be denoted as "Δ_W." In this proposed version, lateral forces are specified at the "Strength Level," and the corresponding displacements are be denoted as Δ_s. When a structure responds to the *Design Basis Ground Motion*, it will experience forces larger than both the working stress and strength level design forces, and will thus likely respond in the inelastic range. The corresponding displacements, denoted as Δ_M, will be several times larger than either Δ_W or Δ_s. In the current version of the Recommendations, this amplified displacement, Δ_M, is estimated as $3(R_W/8)\Delta_W$.

This section states that wherever the Code requires the use or consideration of seismic building drifts, the displacements used or considered must be the drift, Δ_M, that would occur during the design basis earthquake. In other words, any provision involving drift (such as drift limits, seismic separations, deformation compatibility, EBF link rotations, SMRF joint rotations) requires that the drift value used be representative of the drift that would occur when the structure responds elastically or inelastically to a 10 percent in 50-year earthquake. This section defines the terms Δ_s and Δ_M, prescribes how these drifts are to be computed and prescribes drift limitations for the structure.

The user will note that the term Δ_M is used throughout the General Requirements and Materials sections wherever previous versions of the Code cited the "$3(R_W/8)$"

amplification of the working stress level displacements. The proposed version now simply cites "Δ_M" and leaves the definition of this level of displacement to the General Requirements.

UBC §1645.9.1 Determination of Δ_s **.** The drifts, Δ_s, are computed from the lateral displacements of the structure when loaded with the Strength Level *Design Seismic Forces*. These may be computed using the equivalent static force method or the dynamic analysis procedures. Regardless of whether the structure is designed using Allowable Stress Design or Strength Design, the displacements must be computed using the Strength Design load combinations. Also, the mathematical model of the structure used in the calculation of drift must comply with the requirements of UBC UBC §1645.1.2.

UBC §1645.9.2 Determination of Δ_M**.** The inelastic drift, Δ_M, may be approximated by factoring the design level drift, Δ_s, using equation 45-19 (which reads $\Delta_M = 0.7 R_d R_O \Delta_s$).

Equation 45-19 provides a reasonable approximation of the maximum inelastic response displacement for regular buildings. This relation is based on the accumulated research of the past 30 years and on the intent (for the present time) to maintain parity with the current drift limitation requirements. Figure 16-3 illustrates the terms Δ_s and Δ_M.

Background. Nearly 30 years ago, Newmark postulated that the maximum inelastic response displacement (or the "inelastic drift") could be reasonably predicted as the drift of an elastic structure (or the "elastic drift") subjected to the same ground motion. Referring to Figure 16-3, Newmark would state that the inelastic displacement may be approximated by the displacement Δ_E in the figure. Newmark's statement can also be represented by the equation:

$$\Delta_M = 1.0R\ \Delta_s$$

Where R is the total response reduction factor. Over the past 30 years, extensive research on this topic has shown that it is a reasonable approximation, but that for some structures the inelastic drift will be significantly larger than the elastic drift and for other structures, the inelastic drift will be less than the elastic drift. Miranda and Bertero (EERI Earthquake Spectra, Vol 10, No. 2, 1992) provide a summary of this research.

The Seismology Committee selected the value of 0.70 in Formula 45-19, in lieu of Newmark's value of 1.0, as a better "average" representation of the inelastic drift. Thus, while some structures will see less and some more total drift, the 0.70 factor provides a reasonable across-the-board approximation for the types of buildings designed under these provisions.

This section permits the alternative calculation of Δ_M by nonlinear time-history analysis. Where, in the judgment of the engineer, the accurate calculation seismic displacements is needed, nonlinear time-history analysis will provide a more reliable estimation of the inelastic drift than does Formula 45-19. This more sophisticated level of analysis may be prudent when stability, p-delta effects, building separations, magnitude of inelastic joint or plastic hinge rotations, or the effect of drift on portions not part of the lateral system (deformation compatibility) are deemed to be critical values to know for the overall performance and design of the building.

This section requires that analysis used to compute Δ_M consider P-delta effects. This means that if UBC §1645.1.3 requires a P-delta analysis, or if the engineer determines that a P-delta analysis is prudent, the computed value of Δ_M must include displacements associated with secondary P-delta effects. P-delta effects at Δ_M should use forces (story shears) in the inelastic (push-over) structure at the Δ_M deformation.

UBC §1645.10.2 Story Drift Limits. The story drift limits of 0.020 and 0.025 times the story height, combined with the 0.70 factor in Formula 45-19, provide approximately equal stiffness for buildings designed to previous versions of these provisions, and are comparable to lateral stiffness required in the 1994 version of the NEHRP provisions.

While these drift limits provide reasonable parity with previous code requirements, several factors are worthy of note:

1. This parity is achieved primarily for structures designed using the equivalent static force method. Parity may not be obtained for structures analyzed using dynamic analysis techniques.
2. A story drift ratio of 0.025 radian corresponds to a story drift of 3.9 inches for a 13-foot story height. This level of interstory displacement is large and may pose extreme difficulties when attempting to comply with drift related requirements such as deformation compatibility. For example, the deformation compatibility requirements for precast cladding panels require the connectors be capable of surviving a Δ_M level story drift, and the panel joint dimensions are typically based on Δ_M level drifts. The current version produces $3(R_W/8)$ level drift ratios of about 0.015 radian, and much of the industry standard details have been developed for this level of drift.
3. Engineers may elect to invoke stricter drift limits for the structural design, or may utilize more sophisticated methods of analysis such as nonlinear time-history analysis, to determine the Δ_M level displacements.
4. If an engineer elected to prove compliance with the drift limits by nonlinear analysis, and "pushed" the design to the limits of the Code, the design may prove unsatisfactory. An inelastic drift ratio of 0.025 radian is, as stated above, extremely large and represents possibly the physical drift capacity of many

> structural systems. Given that the *Design Basis Ground Motion* represents an average hazard for a relatively large geographic area, and given that a significantly larger earthquake is possible for a given site, using advanced analysis methods to push the liberal drift limit of 0.025 may not be prudent: If the structures lateral drift capacity is exhausted for the design basis ground motion, a larger earthquake may produce catastrophic results. A maximum inelastic drift ratio of 0.010 to 0.015 has for some time been considered a conservative value (ATC-03 and Tri-Services Manual). This level of story drift would be a more appropriate limit when sophisticated analysis techniques such as nonlinear time-history are used.

In summary, the drift limits of this section are appropriate for structures analyzed using the equivalent static force and response spectrum analysis techniques. However, the engineer must determine if the resulting story drifts can be sustained by all structural and nonstructural elements. Also, the engineer must treat these drift limits with caution when more sophisticated analysis techniques are used.

UBC §1646 Dynamic Analysis Procedures

UBC §1646.1 General. This section provides minimum requirements, limitations, and guidelines for performing dynamic analysis. Refer to the Commentary to the current Recommendations where the Provisions are unchanged.

UBC §1646.2 Ground Motion. The analyses are completed using an elastic design response spectrum that is not reduced by an R factor. The maximum inelastic displacements for a structure with a fundamental period in the velocity or displacement controlled region of the response spectrum will be approximately equal to the elastic displacement. Hence, the use of an unreduced spectrum gives displacements that should approximate the actual inelastic displacements. See UBC §1646.5.4, Reduction of Elastic Response Parameters for Design, for structures with a fundamental period in the acceleration controlled region of the spectrum.

The default spectrum shape is shown in Figure 16-4. The ground response parameters C_a and C_v, which quantify the effects of ground motion as a function of the site seismic coefficient S_A to S_F, the seismic zone factor Z, and the near source factor N_a or N_v, are used to construct the spectral ordinates.

UBC §1636.3 Mathematical Model. The model should use stiffness properties that more accurately reflect the in-service condition such as the cracked section stiffness properties set forth in UBC §1645.1.2.

UBC §1646.5.1 Response Spectrum Representation and Interpretation of Results. The results of the response spectrum analysis are based on an unreduced spectrum and a linear elastic structural model. The results are exactly equal to the results that would be obtained if the structure remained completely elastic. Hence, the response results are termed "Elastic Response Parameters," which may be reduced to estimated inelastic response parameters for design in accordance with UBC §1646.5.4.

UBC §1646.5.4 Reduction of Elastic Response Parameters for Design. The Elastic Response Parameters (ERP) are reduced to estimated inelastic response parameters by proportioning to base shear. For regular structures, the response parameters may be reduced to 80 or 90 percent of the corresponding parameters determined using the base shear from the static method as an inducement to use elastic dynamic analysis. Elastic dynamic analysis gives a better representation of the initial distribution of the elastic forces in the structure. This reduction is not available to irregular structures as the inelastic force distribution may be very different from the elastic force distribution as yielding occurs in the structure. If the site specific design spectrum is significantly higher than the default spectrum constructed in accordance with Figure 16-4, the ERP may not be reduced to less than the elastic base shear based on the site specific spectrum divided by R.

Short period structures with a fundamental period in the acceleration controlled region of the spectrum usually have inelastic displacements greater than the displacement obtained if the structure remained linearly elastic. Hence the reduction factor R, should not be constant over the period range of the spectrum, but rather should depend on the period of interest. If the ERP were reduced to inelastic response parameters only by scaling the base shear to that obtained by dividing by the corresponding R factor, then the R factors would need to be varying over the entire acceleration controlled period range or be adjusted by another factor. The spectrum described by the static base shear has a constant acceleration from $T=0$ sec. to $T=T_S$ sec., the acceleration controlled range of the spectrum, rather than a varying acceleration from $T=0$ to $T=T_a$. By using the static base shear as the basis for the reduction, one is essentially correcting the R factor in an adequate manner in the period range from $T=0$ sec. to $T=T_a$ sec.

UBC §1646.6.1 Time-History. A new section has been added that provides explicit requirements for dynamic time-history analysis. Separate requirements have been provided for elastic and nonlinear time-history analysis. For both types of analyses, the same very specific procedures have been provided for selecting and scaling ground motion time-histories that are to be used in such analyses. The selection and scaling procedures are based upon those required for seismic isolation, except that the range of period match of ground motion response spectra is greater because of the greater importance of higher mode effects in fixed base structures.

Prediction of ground motion histories is an inexact science. The requirement of using histories from at least three recorded earthquakes allows for averaging the effects of event uniqueness. Ground motion time histories are highly dependent on the characteristics of the earthquake source, travel path of the motion, and site characteristics, and vary widely.

Design spectra, such as the one in Figure 16-4, are usually created from large suites of ground motion data, which tends to average or smooth the energy content across the entire spectrum of frequencies and create characteristic shapes. Typical histories are rich in energy at some frequencies and deficient in energy at other frequencies. The response spectra computed from these histories are also rich in energy at some frequencies and deficient in energy at other frequencies. A suite of histories should be selected such that the histories match the source and site characteristics and are rich in energy across a broad range of frequencies, or at least complement each other across the range of frequencies of interest for the structure such that the high energy content at a certain frequency in the spectrum of one history will offset the deficiency in energy at the same frequency in another history. This will approximate the averaging or smoothing process used to develop characteristic spectral shapes.

Where recorded histories are not available, simulated histories must be used. The energy content of these simulated histories must approximate a spectrum constructed in accordance with Figure 16-4 over the frequency range of 0.2 to 1.5 times the structures fundamental frequency. This is to ensure that the energy content of the simulated histories is sufficient over the frequencies of interest for the structure.

If only three histories are used, then the maximum response parameter must be used for design; if seven or more histories are used, then the average response parameter may be used for design. This is in accordance with the averaging process discussed above.

UBC §1646.6.2 Elastic Time-History Analysis. Elastic time-history analysis (ETHA) with constant modal damping at 5 percent will provide essentially identical results to precisely performed response spectra analysis. A precisely performed response spectra analysis is an analysis in which a more exact modal combination method is used such as the CQC method, and the actual response spectra (not smoothed) of the ground motion time-history used in the time-history analysis is utilized. The primary value in performing an elastic time-history analysis is in getting a better estimation of relative displacement and in-structure acceleration effects. However, there is no real advantage of performing elastic time-history analysis over precisely done response spectra analysis with regard to base shear, member forces, and overall drift. For this reason, no additional reduction in design forces is given for performing elastic time-history analysis when compared to response spectra analysis, and the same scaling back to the static base shear applies to both. It should be noted that this conclusion also applies to nonconstant modal damping. To achieve virtually the same results if one were to use nonconstant modal damping, one would need to use different damping levels of response spectra with each mode consistent with those used in the time-history analysis in order to achieve consistent and comparable results.

UBC §1646.6.3 Nonlinear Time-History Analysis. Because nonlinear time-history analyses require considerable technical judgment in their proper execution, specific guidance has been limited. Instead, a peer review is required. As indicated earlier, the selection and scaling of time-histories to be used in the analysis is similar to that required for base isolation. The selection of the properties and capacities to be used in the dynamic analysis and evaluation, shall be consistent with experimentally measured or substantiated analysis. There should be appropriate reduction in acceptable ductility proportional to the Importance Factor. The story drifts obtained by the nonlinear analysis shall not exceed the limits specified in Sec. 1645.10.

UBC §1647Lateral Force on Elements of Structures, Nonstructural Components and Equipment Supported by Structures

UBC §1647.2 Design for Total Lateral Force. The newly proposed equations for determining the design seismic force F_p for elements, components, and equipment are dependent upon the weight of the system or component (W_p), the component amplification factor (a_p), the horizontal acceleration of the structure for the design ground motion at the point of component attachment to the structure, the component importance factor (I_p), and the component response modification factor (R_p). Also the design force equations are calibrated to a strength design approach, which translates to an approximate 1.4 increase in force levels as compared to those from previous years in which allowable stress design was used. Equations 47-1 and 47-2 are repeated below for reference.

$$F_p = 4.0 C_a I_p Wp \qquad (47\text{-}1)$$

$$F_p = \frac{a_p C_a I_p}{R_p}\left(1 + 3\frac{h_x}{h_r}\right)W \qquad (47\text{-}2)$$

The seismic design force equations presented as Eq. 47-1 and 47-2 in UBC §1647 originated with a study and workshop sponsored by the National Center for Earthquake Engineering Research (NCEER) with funding from the National Science Foundation (NSF) (Bachman et al., 1993). The participants examined recorded acceleration data in response to strong earthquake motions. The objective was to develop a "supportable" design force equation that considered actual earthquake data as well as component location in the structure, component anchorage ductility, component importance, component safety hazard upon separation from the structure, structural response, site conditions, and seismic zone. Also, there was a desire that there be a reasonable transition of design requirements between a ground supported item and an item supported within a structure. Additional studies have further revised the equation to its present form (Drake & Bachman, 1994 and 1995). It is believed that Eq. 47-1 and 47-2 achieves the stated objectives without unduly burdening the practitioner with complicated formulations.

The component amplification factor (a_p) represents the dynamic amplification of the component relative to the fundamental period of the structure (T). It is recognized that at the time the components are designed or selected, the structural fundamental period is not always defined or readily available. It is also recognized that for many components, the component fundamental period (T_p) is usually only accurately obtained by expensive shake-table or pull-back tests. A listing is provided of a_p

values based on the expectation that the component will usually behave in either a rigid or flexible manner. In general, if the fundamental period of the component is less than 0.06 sec, no dynamic amplification is expected. It should be noted that the component amplification factor was effectively 2.0 in the 1994 UBC. It has been increased to 2.5 in this proposal. This new value is based on the NCEER recommendations and provides consistency with ground supported structures where the maximum amplification factor is also 2.5. It is not the intention of the requirements to preclude more accurate determination of the component amplification factor when reasonably accurate values of both the structural and component fundamental periods are available.

The component response modification factor (R_p) represents the energy absorption capability of the component's structure and attachments. Conceptually, the R_p value considers both the overstrength and ductility of the component's structure and attachments. It is believed that in the absence of current research, these separate considerations can be adequately combined into a single factor for nonstructural components. These values are judgmentally determined utilizing the collective wisdom and experience of the responsible committee. In general, the following benchmark values were used:

$R_p = 1.5$ brittle or buckling failure mode expected

$R_p = 3.0$ moderately ductile materials and detailing

$R_p = 4.0$ highly ductile materials and detailing

Minor adjustments from the 1994 NEHRP Provisions have been made in the tabulated R_p values presented in this proposal. The values were adjusted to provide relative parity with the current Provisions.

Eq. 47-2 represents a trapezoidal distribution of floor accelerations within the structure, linearly varying from the acceleration at the ground (C_a) to the acceleration at the roof ($4.0C_a$). The ground acceleration (C_a) is intended to be the same acceleration used as design input for the structure itself and will include site effects.

A lower limit for F_p is set to assure a minimal seismic design force. The minimum value for F_p determined by setting the quantity a_pA_p/R_p equal to $0.7C_a$, is consistent with the current Provisions minimum value of $0.50Z$ times a factor of 1.4 to convert from allowable stress design to strength design.

To meet the need for a simpler formulation, a conservative maximum value for F_p also was set. Eq. 47-1 prescribes the maximum value for F_p determined by setting the quantity a_pA_p/R_p equal to $4.0C_a$. Eq. 47-1 also serves as a reasonable "cutoff" equation to assure that the multiplication of the individual factors does not yield an unreasonably high design force.

The redundancy factor (r) was set equal to unity since the limiting redundancy of nonstructural components had already been accommodated in the selection of R_p factors.

UBC §1649 Nonbuilding Structures

1649.3 Rigid Structures. The primary change to the nonbuilding structure requirement was to convert the force reduction factors from an allowable stress basis to a strength basis. This conversion was effectively accomplished by dividing the existing R_W factors by 1.4. The R factors were also split with the overstrength factor (R_O) taken as 1.8 and the ductility factor (R_d) adjusted to obtain parity with the current values. The R_d values range from 1.2 to 2.0. This compares with a range of 1.2 to 3.4 for building structures. Because limited redundancy of nonbuilding structures had already been accommodated in the R_d factor, the redundancy factor (ρ) is taken as unity.

The minimum value of $0.56C_aIW$ is equivalent to the minimum value of and is obtained as follows:

$$V_{min} = ZI\left(\frac{C}{R_w}\right)_{min} W = ZI(0.40)W$$

$$(V_s)_{min} = 1.4V_{min} = 1.4ZI(0.40)W$$

Since C_a replaces Z in the proposed 1997 UBC base shear equation, substituting C_a for Z yields:

$$(V_s)_{min} = 1.4C_aI(0.40)W = 0.5C_aIW$$

A relative comparison of force reduction factors obtained for nonbuilding structures is presented in Table C49-1. Since it can be seen in the table that considering the strength conversion factor of 1.4, the force reduction factors are directly equivalent. Therefore, any differences in design forces is directly attributable to the difference between C_a and C_v factors in the proposed Provisions and the Z and S factors from the current Provisions.

New Near Source Factors (Tables 16-Q, 16-R, 16-S, and 16-T)

Prior to the 1994 Northridge earthquake, there was a limited number of records recorded close to fault rupture (e.g., within about 5 km of fault rupture). Table 1 lists these records and summarizes the magnitude and earthquake source for each.

Table AppC-1. Pre-Northridge Earthquake Records Close to Fault Rupture

Instrument Name/ Location	Magnitude	Earthquake Name and Year
Pacoima Dam	$M = 6.6$	1971 San Fernando, CA earthquake
Karakyr Point	$M_S = 7.0$	1976 Gazli (Russia) earthquake
El Centro Array Nos. 5,6,7	$M_L = 6.6$	1979 Imperial Valley, CA earthquake
Tabas	$M = 7.4$	1979 Tabas (Iran) earthquake
Site 1	$M_S = 6.9$	1985 Nahanni (Canada) earthquake
Corralitos	$M_S = 7.1$	1989 Loma Prieta, CA earthquake
Cape Mendocino	$M_S = 6.9$	1992 Petrolia earthquake
Lucerne Valley	$M_S = 7.5$	1992 Landers, CA earthquake

After the 1971 San Fernando earthquake, some experts suggested that the over 1.0 g PGA recorded at the Pacoima Dam was an anomaly (i.e., prior to 1971 most engineers considered 0.5g to be a maximum value of PGA). Since 1971, all instruments located on rock or stiff soil sites close to fault rupture of large-magnitude earthquakes have recorded high peak ground accelerations ranging from 0.6g (Corralitos, 1989 Loma Prieta earthquake) to about 2.0g (Cape Mendocino, 1992 Petrolia earthquake. In some cases, large peak accelerations are accompanied by strong velocity pulses that can significantly affect the response of structures at long periods. Such pulses were recorded along the El Centro Array during the 1979 Imperial Valley earthquake. The instruments of this array are located at about 3 km intervals along a line that runs across and perpendicular to the Brawley and Imperial faults. During the 1979 Imperial Valley, earthquake these faults ruptured and very strong, long-period pulses were recorded at array sites close to fault rupture in the direction perpendicular to the plane of fault rupture.

The number of records close to fault rupture approximately doubled after the 1994 Northridge earthquake. Table AppC-2 summarizes ground shaking data from the 1994 Northridge earthquake recorded at sites 15 km or less from fault rupture (using the definition of distance from site to fault rupture proposed for the 1997 UBC). Six of these near-source sites are located over the plane of fault rupture, as indicated by a 0 km distance to fault rupture. The average PGA for these sites is over 0.7g and the average acceleration response at a period of 1.0 seconds is typically over 0.9g. In contrast, the current spectrum anchored to an effective peak ground acceleration

(EPA) of $Z = 0.4g$ has 1-second acceleration response of 0.4g for rock sites and about 0.6g for soil sites. As Table AppC-2 illustrates, response decreases with distance and at about a 10 km distance from fault rupture response values are approximately consistent with the current spectrum (anchored to an EPA of $Z=0.4g$).

Table AppC-2. *1994 Northridge Earthquake Records (Sites <= 15 km of Fault Rupture)*

Instrument Station Information		Peak Response		Distance to Fault Rupture (km)
Name	Site	PGA (g)	$SA_{1.0}$ (g)	
Jensen Filter Plant - Generator Bldg.	N/A	0.98/0.56	> 0.90	0
Newhall - County Fire Station	Soil	0.63/0.61	> 0.90	0
Rinaldi Receiving Station	Soil	0.85/0.48	> 0.90	0
Sylmar Converter Station (East)	Rock	0.79/0.45	> 0.90	0
Sylmar Converter Station	Soil	0.90/0.61	> 0.90	0
Sepulveda VA Hospital	N/A	0.94/0.74	0.7	0
Sylmar County Hospital	Soil	0.91/0.61	0.75	2
Arleta - Nordhoff Ave. Fire Station	Soil	0.35/0.29	0.6	7
Pacoima - Kagel Canyon	Rock	0.44/0.30	0.45	9
Castaic - Old Ridge Route	Rock	0.59/0.54	0.35	10
Lake Hughes 12A	Soil	0.26/0.18	0.35	12
Energy Control Center	Rock	0.23/0.21	0.4	12
Santa Susana -ETEC	N/A	0.29/0.23	0.25	14

The most significant earthquake to effect a modern urban region occurred in Kobe Japan in early 1995. Over 5,000 people were killed, 35,000 injured and 300,000 left homeless. More than 150,000 buildings were destroyed by the earthquake and ensuing fire. The Hyogo prefectural government has estimated approximately (US) $100 billion of direct damage, primarily due to building loss. Ground shaking in Kobe was intense, but of relatively short duration and typical of ground motion recorded close to fault rupture. Soil effects contributed to the intense shaking in certain areas, but the primary cause of strong ground shaking felt throughout Kobe was the proximity of the city to fault rupture. Near-source ground shaking records from Kobe contained a few very strong long-period pulses, similar to near-source records from other earthquakes such as Northridge. Like Northridge, these pulses were also typically much stronger in the direction of shaking normal to the plane of fault rupture (Somerville 1996).

It is estimated that prior to the earthquake there were about 700,000 buildings in Kobe and other nearby cities of the Hyogo Prefecture and that about 375,000 of these buildings were located within 5 km of fault rupture (Kircher, 1996). The damage surveys by Architectural Institute of Japan (AIJ, 1995) reported that 144,032 of the approximate 700,000 buildings collapsed or were severely damaged. Of particular interest, about 90 percent (i.e., 131,355 of the 144,032) of the buildings that collapsed or were severely damaged were located within 5 km of fault rupture. In contrast, collapsed buildings in Osaka account for only about 1 percent of all buildings that collapsed during the earthquake, even though Osaka is located about 10 km to 30 km from Kobe and contains an even greater number of the same type of buildings as those destroyed in Kobe.

Ground motion records indicate that the factor is best described by distinguishing between the short-period (acceleration) and long-period (velocity) domains since the effect is substantially greater at longer periods. This distinction is made through the use of the N_a and N_v factors. The values for the N_a factor are based on the ratio of 0.3 second response predicted by the median estimates of ground motion attenuation functions to the 0.3 second response of the UBC spectrum for stiff soil anchored at an EPA of $Z=0.4g$. The N_v factor is derived similarly for the 1.0 second period.

The average of two well-known attenuation functions was used to estimate 1.0-second response for a stiff soil site: Boore, Joyner and Fumal (1993), taken as the average prediction for Site "B" and Site "C" soils, and Sadigh et al. (1986), taken as the average prediction for strike-slip and reverse-slip source mechanisms. In both cases, median (50 percentile) predictions of the "random" horizontal component were used to estimate near-source ground shaking. As such, the near-source factor represents typical horizontal ground shaking and is not intended to account for higher levels of shaking that occur due to the inherent variability of ground motion. Similarly, the near-source factor is not intended to bound ground motion in the strongest possible (e.g., fault normal) direction of shaking.

Evaluation of the near-source factor requires knowledge of the location, dip angle (for nonvertical faults), maximum moment magnitude potential and slip rate (activity level) of faults. The location and dip angle are required to determine the geometry of the fault plane and distance form the site to surface projection of the fault rupture plane. The surface projection of vertical fault planes (e.g., strike-slip faults) is simply a line along the surface. The surface projection of nonvertical fault planes has some nonzero width based on the dip angle and fault depth. The method of determining distance from a nonvertical fault in accordance with these Requirements is illustrated in Figure 1. Portions of faults deeper than 10 km need not be included in the determination of the surface projection of the fault rupture plane. Fault location may be determined from maps of known faults prepared by the United States Geologic Survey (USGS) or the California Department of Conservation Division of mines and Geology (CDMG). For example, a detailed map of faults in California has been recently updated by Jennings (CDMG 1994).

The maximum magnitude, M, and slip rate, SR, are used to classify seismic source types. Type A sources are faults that have a moment magnitude potential of $M>=7.0$ and a slip rate SR equal to or greater than 5 mm/year. These types of faults are considered to be active and capable of producing large magnitude events. Most segments of the San Andreas fault would be classified as a Type A fault. Type C sources are faults that have a moment magnitude potential of M less than 6.5 and a slip rate of SR less than or equal to 2 mm/year. These types of faults are considered to be sufficiently inactive and not capable of producing large magnitude events such that potential near-source ground shaking effects can be ignored. Most faults outside of California are Type C. Type B sources are all faults that are not either Type A or Type C, and include most of the faults in California. Magnitude and slip rate information for faults may be obtained from the USGS or the CDMG. For example, a recent compilation of data on all faults in the western United States has been made by the USGS as a part of Project 97, a national seismic hazard mapping project (Frankel et al., 1996).

New Seismic Coefficients

Tables 16-Q and 16-R specify seismic coefficients, C_a and C_v, respectively, in terms of the seismic zone factor, Z, the near-source factors, N_a and N_v, and the soil profile type. For construction of design response spectra, C_a is the EPA and C_v is the value of acceleration response at a 1.0-second period. In Tables 16-Q and 16-R, the seismic zone factor, Z (or the product, ZN_a or ZN_v, for sites in Seismic Zone 4 that have a near-source factors N_a or N_v greater than 1.0) represents the shaking intensity for rock site conditions (Soil Profile Type B). The tables specify seismic coefficients for Soil Profile Types C, D and E that include the effect of soil amplification on site response. Seismic coefficients for Soil Profile Type F are not provided and must be determined by a site-specific hazard analysis. Soil Profile Type A (hard rock) is only appropriate for classification of sites in the eastern United States.

UBC §1662 Site Categorization Procedure

The site categorization procedure in the 1994 UBC is based on work completed in the 1970's. The "site factors" were constants independent of level of ground shaking. Sites were characterized by a somewhat subjective qualitative description. The proposed site characterization procedure in this code change is based upon more recent data. The "site factors" are nonlinear and are a function of the expected input acceleration, i.e, the zone factor, and the site soil characterization, i.e., the site factor. The site characterization is based on easily obtainable quantitative geotechnical parameters rather than qualitative descriptions. Soil Profile Type A is a very hard rock characterized by the hard granite formations underlying the eastern states. Most of the rock in the western states will be Soil Profile Type B.

Because of the extensive strong motion instrumentation program undertaken since the 1971 San Fernando earthquake, much more data has become available for a wide range of ground shaking and site characteristics. In particular, data from the 1985 Mexico City and 1989 Loma Prieta earthquake have contributed to the development of the new site categorization procedure and coefficients. The site characterization procedure was adapted from the procedure in the *1994 NEHRP Recommended Provisions for Seismic Regulations for New Buildings*, which was developed at the 1992 NCEER/SEAOC/BSSC Site Response Workshop.

Prior to the 1985 Mexico city earthquake, studies had shown that, in general, earthquake motions were attenuated as they propagated from rock to soil for rock accelerations greater than 0.1g. There was insufficient data for soft soils so the relationships were extrapolated from firm soil data. Data from the 1985 Mexico City earthquake and the 1989 Loma Prieta earthquake show that the motions can be significantly amplified, not attenuated, for soft sites and moderate accelerations. The largest amplification typically occurs at the natural period of the soft soil deposit. For example, in the Mexico City shaking, the rock acceleration was amplified four times by the soft clay deposit at $T=0$ sec. while the spectral amplitude at approximately $T=2$ seconds indicated an amplification of 15-20 times the rock acceleration. Data from the Loma Prieta earthquake also indicate an amplification at long-period spectral amplitudes of three to six times, and the largest amplification occurred at the natural period of the soil deposit. However, at stronger levels of ground shaking, this later data corroborates the results of previous studies that the motion attenuates in soft soils as the strength of the soil is exceeded and the soil behaves in a highly nonlinear manner.

As the ground motion propagates from rock through the overlying soil medium, the rock motion is amplified or attenuated as discussed above. Nonhomogeneous layered sites with soft layer between the rock and the stiffer soils complicate the problem. The parameter necessary to characterize the site and determine analytically the amplification or attenuation and frequency modification are the thickness of the soft and stiff soil layers, the shear wave velocities of the rock and soil layers, the soil/rock impedance ratio, the layering and properties of the soil layers including modulus and damping properties. The amplification ratio is a function of the rock/soil impedance contrast ratio (which is a function of shear wave velocity), the soil damping, and the ratio of the frequencies of the layers. The impedance contrast must consider the whole soil profile and thus both the soft and stiff soil layers play a role in determining the damping and the amplification (or attenuation). The maximum amplification generally occurs at or close to the natural frequency of the soil deposit, but may occur at the natural frequency of the soft layer, the natural frequency of the whole soil deposit, or some other frequency. In all cases, the shear wave velocity of the rock and soil deposit has a dominant role. Hence, the site is characterized primarily by the shear wave velocity. In shallow or moderate soil depth sites where the soil depth is less than 30 m (100 ft), the entire soil deposit is important to the overall response. In deep soil (greater than 30 m) sites having stiff soil profiles, the motion at the surface tends to be dominated by the soils in the upper 30-50 meters; hence, the site can be characterized by the average shear wave velocity in the upper 30 m (100 ft). Deep soil (greater than 30 m) sites having thick (35 m) soft-medium stiff clays, or highly plastic or organic clays and peats of any thickness, cannot be characterized in this manner and must be evaluated on a site specific basis. Similarly, sites having liquefiable soils, e.g. loose sands, or highly sensitive soils that may loose strength during an earthquake, e.g. bay muds, must be evaluated on a site specific basis.

Shear wave velocities are not available in many cases and the size of the project may not warrant the expense of obtaining the shear wave velocities. Hence, alternate methods of site characterization are also included. These methods use the Standard Penetration Resistance, N, for the soil deposit as a whole, or alternatively, the Standard Penetration Resistance (SPT), N, for cohesionless soil layers and undrained shear strength, S_u, for cohesive soil layers. These soil parameters are generally obtained from field tests done during site borings and are inexpensive. The correlation of site amplification with these alternative parameters is more uncertain that with the average shear wave velocity. For important or large projects, the site characterization should be obtained from shear wave velocity data.

The procedure for characterizing a site is as follows:

Step 1. Check for the four categories of soils set forth in Soil Profile Type F requiring site specific evaluation. If the site soils correspond to any of these categories, characterize the site as Soil Profile Type F and conduct a site specific evaluation.

Step 2. Check the total thickness of soft clay layers. Soft clay is defined as a clay with an undrained shear strength, S_u, less than 25 *kPa* (S_u < 500 psf), a water content, w, equal to or greater than 40 percent, and a Plasticity Index (PI) greater than 20. If the total thickness of the soft clay layers is greater than 3 m (greater than 10 ft), characterize the site as Soil Profile Type E.

Step 3. Depending on the site geotechnical data available, for the top 30 m (100 ft) determine either the average shear wave velocity, v_S, in accordance with Equation 62-1; the average SPT value, $\bar{N}$, for the site as a whole in accordance with Equation 62-2; or the average SPT, $\bar{N}_{CH}$, for the cohesionless layers in accordance with Equation 62-3 and the average undrained shear strength, S_u, in accordance with Equation 62-4. Cohesionless layers are those for which the PI is less than 20; cohesive layers have a PI greater than 20. Select the appropriate soil profile from Table 16-J.

See the *1994 NEHRP Recommended Provisions for Seismic Regulations for New Buildings* Section 1.4.2, Site Coefficients, and Commentary Section 1.4.2 and Appendix 1D for further elucidation.

References

Architectural Institute of Japan (AIJ), 1995, "Preliminary Reconnaissance Report of the 1995 Hyogoken-Nanbu Earthquake," English Edition, Tokyo Japan.

Bachman, R. E., R. M. Drake, and P. J. Richter. 1993. *1994 Update to 1991 NEHRP Provisions for Architectural, Mechanical, and Electrical Components, and Systems*, letter report to the National Center for Earthquake Engineering Research, February 22.

Boore, D.M., W.B. Joyner and T.E. Fumal, 1993, "Estimation of Response Spectra and Peak Accelerations from Western North American Earthquakes: An Interim Report," U.S. Geological Survey Open File Report 93-509.

Drake, R. M. and R. E. Bachman. 1996. "NEHRP Provisions for 1994 For Nonstructural Components," ASCE Journal Of Architectural Engineering, March, 1996.

Drake, R. M. and R. E. Bachman. 1995. "Interpretation of Instrumented Building Seismic Data and Implications for Building Codes," in Proceedings of 1995 SEAOC Annual Convention.

Drake, R. M. and R. E. Bachman. 1994. 1994 NEHRP Provisions for Architectural, Mechanical, and Electrical Components, 5th United States National Conference on Earthquake Engineering.

California Department of Conservation Division of Mines and Geology (CDMG), 1994, *Geologic Data Map No 6*, prepared by C. Jennings, CDMG, Sacramento California, 1994.

Federal Emergency Management Agency (FEMA), 1995, *NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings*, FEMA 222A/223A, Federal Emergency Management Agency, Washington DC, May 1995.

Frankel, A., C. Mueller, T. Barnhard, D. Perkins, E.V. Leyendecker, N. Dickman, S. Hanson, M. Hopper, 1996, "Interim National Seismic Hazard Maps: Documentation," U.S. Geological Survey, Denver Colorado, draft: January 18, 1996.

Kircher, C.A., N.C. Donovan, L. Mualchin, J. Ragsdale, K.L. Benuska, M. Lew, V. Berger and D. Magee, 1993, "An Acceptable Method for Characterizing Seismic Hazard," *Proceedings of the 62nd Annual Convention*, Structural Engineers Association of California, Sacramento California, 1993.

Kircher, C.A., "The Kobe earthquake: Ground Shaking, Damage and Loss," *ASCE Structures Congress '96*, April 15-19, 1996, American Society of Civil Engineers, Washington DC.

Rinne and Dobry, 1992, Memorandum to Sharpe (Chairman TS-12, NEHRP PUC), dated December 11, 1992, "Preliminary Site Response Recommendations," based on NCEER/SEAOC/BSSC Site Response Workshop, University of Southern California, November 18-20, 1992.

Sadigh, K., J. Egan, R. Youngs, 1986, "Specification of Ground Motion for Seismic Design of Long Period Structures" (abs.), *Earthquake Notes*, 57, 13.

Somerville, P., "Forward Rupture Directivity in the Kobe and Northridge Earthquakes, and Implications for Structural Engineers," *Seventh U.S.-Japan Workshop on Improvement of Structural Design and Construction Practices, Lessons Learned from Kobe and Northridge*, January 18-20, 1996, Kobe Japan.

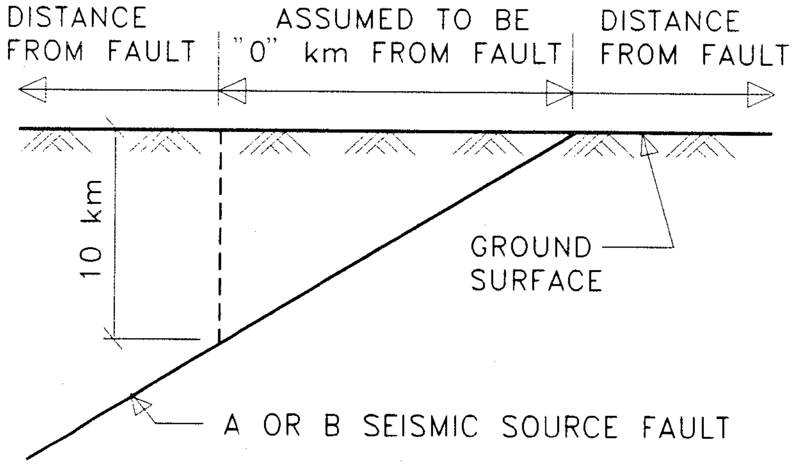


Figure AppC-1. *Determination of distance from nonvertical fault for use in establishing near source factor*

Addendum

On September 10, 1996, at their Annual Business Meeting in St. Paul, Minnesota, the ICBO membership approved, with minor differences, SEAOC's Strength Code Change as presented in Appendix C. It is important to note that in the approved code change, the force reduction coefficient is a single R factor whose value is identical to the product of the R_d and R_0 factors found in Appendix C for each structural system.

Also, the values of Ω_o specified in Tables 16-N and 16-P in the approved code change are identical to the values determined by equation 45-13 of Appendix C. Therefore, the differences in terminology noted below do not result in any change in design or detailing forces.

Differences Between the Proposed Strength Design Code Change and the Approved 1997 UBC

Terminology

Proposed	*Approved 1997 UBC*
$R_d R_o$	R
$\Omega_o = 1.1 R_o$	Ω_o from Tables 16-N and 16-P
$V = \dfrac{C_v I}{R_d R_o T}$	$V = \dfrac{C_v I}{RT}$

Revised Simplified Static Limitations and Force Equations

UBC §1644.8.2 *Revise as follows:*

UBC §1644.8.2 Simplified Static. The simplified static lateral force procedure set forth in Section 1645.2.3 may be used for the following structures of Occupancy Categories 4 or 5.

1. Buildings of any occupancy (including single-family dwellings) not more than three stories in height, excluding basements, which use light-frame construction.
2. Other buildings not more than two stories in height excluding basements.

UBC §1645.2.3.2, 16345.2.3.3 and 1645.2.3.4. *Revise as follows:*

UBC §1645.2.3.2 Base Shear. The total design base shear in a given direction shall be determined from the following formula

$$V = \frac{3.0C_a}{R}W \qquad \text{Eqn. 45-11}$$

where the value of C_a shall be based on Table 16-Q for the Soil Profile Type. When the soil properties are not known in sufficient detail to determine the Soil Profile Type, Type S_D shall be used in Seismic Zones 3 and 4 and Type S_E shall be used in Seismic Zones 1, 2A, and 2B. In Seismic Zone 4, the near-source factor N_a need not be greater than 1.3 if non of the following structural irregularities are present: Type 1, 4, or 5 of Table 16-L and Type 1 or 4 of Table 16-M.

UBC §1645.2.3.3 Vertical Distribution. The forces at each level shall be calculated using the following formula:

$$F_x = \frac{3.0C_a}{R}W_i \qquad \text{Eqn. 45-12}$$

where the value of C_a shall be as determined in Section 1645.2.3.2.

UBC §1645.2.3.4 Applicability. Sections 1645.1.2, 1645.1.3, 1645.2.1, 1645.2.2, 1645.5, 1645.9, 1645.10, and 1646 shall not apply when using the simplified procedure.

> ***Exception:*** For buildings with relatively flexible structural systems, the building official may require consideration of P-Δ effects and drift in accordance with Sections 1645.1.3, 1645.9, and 1645.10. Δ_s shall be prepared using design seismic forces from Section 1645.2.3.2.

Where used, Δ_m shall be taken equal to 0.01 times the story height of all stories. In Section 1648.2.9, Formula 48-1 shall read $F_{px} = \frac{3.0C_a}{R}W_{px}$ and need not exceed $1.0C_aW_{px}$, but shall not be less than $0.5C_aW_{px}$. R and Ω_o shall be taken from Table 16-N.